A DATABASE FOR COMPOSITE COLUMNS

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A_c = area of concrete, in.^{2}

A_g = gross area of steel shape or tube, in.^{2}

A_r = area of reinforcing steel, in.^{2}

A_{rE} = area of reinforcing steel within $2h_E$ region, in.^{2}

A_{ri} = area of one reinforcing bar, in.^{2}

A_m = area if reinforcing bar, in.^{2}

A_{mi} = area of one reinforcing bar within $2h_n$ region, in.^{2}

A_s = area of steel shape or tube, in.^{2}

A_w = area of web of steel shape, in.^{2}

b = length of longer side of rectangular steel tube, in.

b_f = steel section flange width, in.

B_1, B_2 = moment amplification factors

B/t = ratio of longer dimension-to-thickness for rectangular steel tube

c_1, c_2, c_3 = numerical coefficients for composite sections

c_r = average distance from compression face to longitudinal reinforcement in that face and distance from tension face to longitudinal reinforcement in that face, in.

C_{rc} = distance from compression face to longitudinal reinforcing steel in that face, in.

C_{rt} = distance from tension face to longitudinal reinforcing steel in that face, in.

C_m = moment distribution function

COV = coefficient of variation

D = steel section depth, in.

d, D = outside diameter of circular steel tube, in.

D/t = diameter-to-thickness ratio for circular steel tubes

e = eccentricity of applied load, in.

e/D = eccentricity-to-diameter ratio for circular CFT

= eccentricity-to-depth ratio for SRC

e_i = distance from reinforcing bar to bending axis considered, in.

E_c = elastic modulus of concrete, ksi

E_{cm} = secant modulus of concrete, ksi

EI_{eff} = effective moment of inertia rigidity of composite section, kip-in.^{2}

$(EI)_e$ = effective bending stiffness, kip-in.^{2}

E_m = modified elastic modulus, ksi

E_r = elastic modulus of reinforcing steel, ksi

E_s, E = elastic modulus of steel shape or tube, ksi

f'_c = compressive cylinder strength of concrete,ksi

f_{cd} = design value of concrete compressive strength, ksi

f_k = intermediate value for calculating slenderness, ksi

f_{rd} = design value of steel yield stress for reinforcement, ksi

f_{yd} = design value of steel yield stress for shapes and tubes, ksi

F_{cr} = critical stress, ksi

F_{my} = modified yield stress, ksi

F_y = yield stress of steel shape or tube, ksi

F_{yr} = yield stress of reinforcing steel, ksi

h_E = distance from centroidal axis to neutral axis for point E

h_n = distance from centroidal axis to neutral axis, in.

h_1 = overall thickness of composite cross-section in the plane of buckling, in.

width of composite cross-section perpendicular to plane of bending, in.

h_2 = width of composite cross-section parallel to plane of bending, in.

I_c = moment of inertia of concrete, in^4.

I_r = moment of inertia of reinforcing steel, in.^{4}

I_s = moment of inertia of steel shape or tube, in.^{4}

k = effective length factor

= second order moment factor

K_e = a correction factor that should be taken as 0.6.

l = laterally unbraced length of member, in

M_c = available flexural strength, kip-in.

= $\phi_b M_n$

M_b = balanced moment, kip-in.

M_E = moment capacity for neutral axis located h_E from centroid axis, k-in.

ΔM_E = plastic moment of cross-section resulting from region $2h_E$

M_{lt} = required flexural strength due to lateral translation frame, k-in.

M_{max} = maximum internal moment, k-in.

M_n = nominal moment resistance, k-in.

M_{nt} = required flexural strength assuming no lateral translation, k-in.

M_p = approximated plastic moment capacity, k-in.

M_r = required flexural moment, kip-in.

M_{pl} = plastic moment capacity, k-in.

M_{pn} = plastic moment of cross-section resulting from region $2h_n$

M_{Sd} = design moment, k-in.

M_u = ultimate moment, k-in.

M_{ux} = required flexural strength about the x-axis, including second order effects,k-in.

M_{nx} = nominal moment strength for bending about the x-axis, k-in.

M_{uy} = same as M_{ux} except referred to the y-axis.

M_{ny} = same as M_{nx} except referred to the x-axis.

M_1, M_2 = applied end moments, k-in.

N_E = axial load when neutral axis located at h_E

N_{pm} = axial force resistance of concrete portion of cross-section, kips

P_c = available compressive strength, kips

= $\phi_b P_n$

P_{cb} = axial compressive strength at balanced moment, kips

P_d = axial dead load, kips

P_E = Euler buckling load, kips

P_{Em} = Euler buckling load for modified properties, kips

P_{exp} = experiment ultimate axial load, kips

P_l = axial live load, kips

P_n = nominal axial resistance, kips

= nominal compressive strength considering the member as loaded by axial compression only in accordance with LRFD-E2, kips.

P_{pl} = plastic axial resistance of composite cross-section, kips

P_{pred} = predicted axial load in data analysis, kips

P_r = required compressive strength, kips

P_{Sd} = design axial load, kips

P_u = ultimate axial load, kips

= required compressive strength, kips

r = radius of gyration of steel section, in.

= corner radius of rectangular steel tube, in.

= ratio of the smaller end moment to the greater moment

r_m = modified radius of gyration, in.

t = thickness of steel tube, in.

t_f = steel section flange thickness, in.

t_w = steel section web thickness, in.

w = unit weight of concrete, lb./ft^3.

Z, Z_s = plastic section modulus of steel shape or tube, in.^{3}

Z_c = plastic section modulus of concrete, in.^{3}

Z_{cE} = plastic section modulus of concrete within $2h_E$ region, in.^{3}

Z_{cn} = plastic section modulus of concrete within $2h_n$ region, in.^{3}

Z_r = plastic section modulus of reinforcing steel, in.^{3}

Z_{rE} = plastic section modulus of reinforcing steel within $2h_E$ region, in.^{3}

Z_{rn} = plastic section modulus of reinforcing steel within $2h_n$ region, in.^{3}

Z_{sE} = plastic section modulus of steel section within $2h_E$ region, in.^{3}

Z_{sn} = plastic section modulus of steel section within $2h_n$ region, in.^{3}

α = slenderness parameter

= imperfection factor

= 0.21 for concrete-filled circular and rectangular hollow sections.

= 0.34 for completely or partly concrete-encased I-section with bending about the major axis of the profile.

= 0.49 for completely or partly concrete-encased I-section with bending about the minor axis of the profile.

β = moment factor

ε = local buckling limiting value

ϕ = resistance factor

ϕ_b = resistance factor for bending

ϕ_c = resistance factor for compression

γ_c = partial safety factor for concrete = 1.5

γ_r = partial safety factor for steel = 1.15

γ_s = partial safety factor for steel = 1.1

η_1 = reduction factor for steel yield stress due to confinement

η_2 = factor for increasing concrete compressive strength due to confinement

η_{10} , η_{20} = intermediate values for confinement calculations

κ = slenderness reduction factor

κ_n = limit slenderness reduction function for considering imperfections.

λ = general slenderness parameter

λ_c = column slenderness parameter

λ_m = modified slenderness parameter

μ = percentage of plastic moment available for resisting applied loads

μ_d = percentage of plastic moment available

μ_k = percentage of plastic moment reserved for imperfection moments

ρ_{ss} = structural steel ratio

SUMMARY

A database of composite column tests was augmented and utilized to evaluate the proposed AISC 2005 provisions. The database consists of column and beam-column steel-concrete columns (or encased, SRC), circular concrete filled tubes (CCFT), and rectangular concrete filled tube (RCFT). Information on material and geometric properties on each specimen was summarized. The database includes 119 SRC columns, 136 SRC beam-columns, 312 circular CFT columns, 198 circular CFT beam-columns, 222 rectangular CFT columns and 194 rectangular CFT beam-columns. The database has a total of 1181 specimens, an addition of 451 specimens over those in the original database (Aho 1996).

The data on each specimen was analyzed and compared with current design provision for composite columns (AISC 1999 and Eurocode 4), and reassessed by the upcoming 2005 AISC specification. The data indicates that the Eurocode gives good predictions for columns and the AISC 2005 method performs very well for beam-columns. For rectangular CFT columns, all three methods predict the ultimate capacity very well. The main improvement for the AISC 2005 method is its ability to handle specimens which have high yield stress and/or high strength concrete.

CHAPTER Ⅰ

INTRODUCTION

1.1 Background

Numerous different structural systems are used today to meet performance or functional requirements in structures. Composite construction is widely used in structural systems to achieve long spans, lower story heights, and provide additional lateral stiffness. Composite construction uses the structural and constructional advantages of both concrete and steel. Concrete has low material costs, good fire resistance, and is easy to place. Steel has high ductility and high strength-to-weight and stiffness-to-weight ratios. When properly combined, steel and concrete can produce synergetic savings in initial and life-cycle costs.

Currently composite floor systems are widely utilized in steel buildings in the form of composite beams and joists/trusses. As compared to composite floor systems, composite columns are still not very popular in the USA, although they are extensively used in Japan and the Far East. There are two basic kinds of composite columns: steel sections encased in concrete (steel-reinforced concrete sections or SRC) and steel sections filled with concrete (concrete filled tubes or CFT). The latter can be either circular (CCFT) or square/rectangular (RCFT) in cross-section. In composite columns additional synergies between concrete and steel are possible: (a) in concrete-filled tubes, the steel increases the strength of the concrete because of its confining effect, the concrete inhibits local buckling of the steel, and the concrete formwork can be omitted; and (b) in encased sections, the concrete delays failure by local buckling and acts as fireproofing while the steel provides substantial residual gravity load-carrying capacity after the concrete fails.

The current design provisions for composite columns come primarily from the Manual of Steel Construction - Load and Resistance Factor Design (LRFD) [AISC 1999]. These provisions are based in rules developed in the 1960s and 1970s (SSRC TG20, 1979) and utilize an approach in which the composite column is turned into an equivalent steel one. This approach has been shown to yield very low reliability indices (Galambos and Sulyok-Selimbegovic; 1994 Leon and Aho, 2000), and a complete reassessment of the design for composite columns has been made in the upcoming 2005 AISC Specification. To support those changes, this thesis had primary three objectives:

1) Augment a composite column database originally developed by Aho (1997) with data primarily from the late 1990s and early 2000s.

2) Utilize the database to assess the robustness of the proposed AISC 2005 provisions for composite columns, and

3) To propose improvements to the 2005 Specification, if deemed necessary.

1.2 **Organization**

The thesis is organized into 5 chapters. Chapter 2 provides a detailed review of current specifications, including the AISC 1999, AISC 2005 and 1994 Eurocode, for composite column and beam-column design. Chapter 3 presents the development of the database, including a brief summary of important parameters for each test. Chapter 4 presents an analysis of the database and results of comparison studies between codes. The results are divided and analyzed by composite columns type (SRC, CCFT, and RCFT) column and beam-columns. Chapter 5 summarizes the results of this research.

Appendix A contains abbreviated versions of the tables that make up the database. The main properties and results of each test are included here. Appendix B includes examples of composite columns design. Finally, Appendix C shows the details of the database through properties and result values of some specimens.

Chapter Ⅱ

LITERATURE REVIEW

The design of composite columns is addressed by a large number of design specifications. In the USA, both the American Institute of Steel Construction (AISC) and the American Concrete Institute (ACI) provide rules for the design of these structural elements. The ACI 318 provisions for composite columns treat these elements basically as a variation of a regular reinforced concrete column and have remained unchanged for many years. Because this thesis is primarily aimed at improving the AISC design provisions, no comparisons are provided to the ACI provisions herein. Mirza et al. (1996) have provided such comparisons recently and that work is not duplicated in this document.

Two versions of the AISC Specifications will be discussed in this work: the old provisions (last issued in 1999) and the new ones that have been approved and will be issued in late 2005. Among foreign specifications, the Eurocode (ENV 1994), the Architectural Institute of Japan (AIJ, 1997), the Building Code of Australia (BCA, 2005), and the New Zealand building code (the NZBC, 1992) standards provide rules for the design of composite columns. As the Eurocode presents the most recent and comprehensive review of composite column design, it was chosen as the third comparison specification for this work. This chapter presents a description of the design specifications (AISC 1999, AISC 2005, and Eurocode 1994) that will be used for the comparison studies.

2. 1 AISC

The AISC design procedure, as introduced in the Manual of Steel Construction - Load and Resistance Factor Design [AISC 1999], is intended to be a compromise between the need for a practical approach and the need to reflect the complex behavior of composite columns. The AISC design method utilizes a modified cross-sectional approach for the composite column; the composite column is designed as an equivalent steel column using the modified properties in place of the steel properties. The extant AISC approach was based on the report "A Specification for the Design of Steel-Concrete Composite Columns" by Task Group 20 of the Structural Stability Research Council [SSRC 1979]. This report recognized that steel-concrete composite compression members probably behave in a very similar manner to ordinary concrete compression members. It was felt that although the SSRC guidelines used the AISC Allowable Stress Design (ASD) Specifications, these guidelines would work for LRFD as well [Galambos and Chapuis, 1980]. ASD specifications for composite columns were never formally recognized by

AISC.

2.1.1 LRFD Steel Column Design (1999)

As the basis for the AISC LRFD design for stability of composite columns, the AISC LRFD steel column design will be reviewed first to account for the similarities and differences between steel and composite design. The strength requirement in load and resistance factor design is stated as:

$$\phi_c P_n \geq P_u \tag{2-1}$$

The design compressive strength for a column is $\phi_c P_n$ where the resistance factor is

$$\phi_c = 0.85 \tag{2-2}$$

and the nominal strength P_n of columns is given by

$$P_n = A_g F_{cr} \tag{2-3}$$

and P_u is a factored design load.

Equation 2-3 states that nominal resistance in compression, P_n is attained when the gross area of the steel (A_g) reaches a critical stress F_{cr}.. The critical stress is a function of the slenderness of the column and steel properties. The column slenderness parameter, λ_c is defined as

$$\lambda_c = \frac{Kl}{r\pi}\sqrt{\frac{F_y}{E}} \tag{2-4}$$

Based on this slenderness, the critical stress for $\lambda_c \leq 1.5$ is

$$F_{cr} = (0.658^{\lambda_c})F_y \qquad (2\text{-}5a)$$

and for $\lambda_c > 1.5c$

$$F_{cr} = \frac{0.877}{\lambda_c^{\,2}} F_y \qquad (2\text{-}5b)$$

λ_c is evaluated for each principal axis, with the axis having the larger slenderness ratio, KL/r, governing the design. In Equation (2-5), the variables are as follows:

A_g = gross area of member, in.^{2}

F_y = specified yield stress, ksi

E = modulus of elasticity, ksi

K = effective length factor

l = laterally unbraced length of member, in

r = governing radius of gyration about the axis of buckling, in

This column design procedure is simple from the design standpoint. The first step is to determine the governing slenderness ratio. From this ratio the critical stress is determined and from the stress, the critical load. By applying the appropriate resistance factor, the design capacity of the column can be determined.

Beam-columns are members that are subjected simultaneously to axial forces and bending moments. Thus, their behavior falls somewhere between that of a pure, axially loaded column and that of a beam with only moments applied. To understand the behavior of beam-columns, it is common practice to look at the response as predicted through an interaction equation between axial loads and moments. For steel beam-columns, AISC uses two straight lines to model the interaction of flexure and compression. The flexural and compressive interaction for uniaxial bending is limited by the following equations:

For $\frac{P_u}{\phi P_n} \geq 0.2$

$$\frac{P_u}{\phi P_n} + \frac{8}{9}\frac{M_u}{\phi_b M_n} \leq 1.0 \tag{2-6a}$$

For $\frac{P_u}{\phi P_n} \leq 0.2$

$$\frac{P_u}{2\phi P_n} + \frac{M_u}{\phi_b M_n} \leq 1.0 \tag{2-6b}$$

For biaxial bending, the interaction equations are

For $\frac{P_u}{\phi P_n} \geq 0.2$

$$\frac{P_u}{\phi P_n} + \frac{8}{9}\frac{M_{ux}}{\phi_b M_{nx}} + \frac{8}{9}\frac{M_{uy}}{\phi_b M_{ny}} \leq 1.0 \tag{2-7a}$$

For $\frac{P_u}{\phi P_n} \leq 0.2$

$$\frac{P_u}{2\phi P_n} + \frac{M_{ux}}{\phi_b M_{nx}} + \frac{M_{uy}}{\phi_b M_{ny}} \leq 1.0 \tag{2-7b}$$

where,

P_u = required compressive strength, kips

P_n = nominal compressive strength considering the member as loaded by axial compression only in accordance with LRFD-E2, kips.

M_{ux} = required flexural strength about the x-axis, including second order effects.

M_{nx} = nominal moment strength for bending about the x-axis, in accordance with LRFD-F1.

M_{uy} = same as M_{ux} except referred to the y-axis.

M_{ny} = same as M_{nx} except referred to the x-axis.

x = subscript relating symbol to strong axis.

y = subscript relating symbol to weak axis.

ϕ_c = resistance factor for compression = 0.85

ϕ_b = resistance factor for flexure = 0.9

The procedure for determining the nominal flexural strength M_n for a steel beam-column is not presented here. This information can be found in Section F1 of the AISC Manual [AISC 1999]. The required moment capacity M_u must account for first and second order moments according to Equation (2-8)

$$M_u = B_1 M_{nt} + B_2 M_{lt} \tag{2-8}$$

where,

M_{nt} = required flexural strength in member assuming there is no lateral translation of frame, kip-in.

M_{lt} = required flexural strength in member as a result of lateral translation of frame only, kip-in.

Provisions for computing B_1 and B_2 are given in LRFD-C1. Alternatively, the elastic second-order moments can be used for M_u.

The moments to be used in Equations (2-6) and (2-7) must include both the member and structure second-order effects. Thus, a first-order analysis without sidesway must be carried out, yielding moments (M_{nt}) to be amplified by B_1. Then a first-order analysis including lateral loads and permitting translation must be carried out. This will yield moments with translation (M_{lt}) to be amplified by B_2. The first part is the only term of the considered for the present study because the specimens used for comparison had no lateral translation at their ends. In this work, a first-order analysis will be used and the amplification factor B_2 is set to 0.0. The amplification factor B_1 is defined as

$$B_1 = \frac{C_m}{(1 - P_u / P_E)} \geq 1.0 \tag{2-9}$$

where

$$P_E = \frac{\pi^2 EI}{(KL)^2} = \frac{A_g F_y}{\lambda_c{}^2} \tag{2-10}$$

and C_m is defined as for compression members not subjected to transverse loading between their supports in the plane of bending,

$$C_m = 0.6 - 0.4(M_1 / M_2) \tag{2-11}$$

C_m is a coefficient based on elastic first-order analysis assuming no lateral translation of the frame. M_1/M_2 is the ratio of the smaller to larger end moments at the ends of the member unbraced length in the plane of bending. M_1/M_2 is positive when the member is bent in reverse curvature and negative when the member is bent in single curvature. This term accounts for the moment distribution in the member. For compression members subjected to transverse loading between their supports, the value of C_m is 0.85 for members whose ends are restrained and 1.0 for members whose ends are unrestrained.

2.1.2 AISC LRFD Composite Column Design (1999)

The AISC composite column design provisions are subject to the following limitations:

- The area of the steel section must be at least 4 percent of the composite cross section.
- The SRC section must be reinforced with longitudinal steel bars and transverse ties.
- Transverse ties must be spaced less than 2/3 of the least dimension of the cross-section.
- Each area of longitudinal reinforcement and transverse ties must be at least 0.007 in^2. / in. of bar spacing.
- Clear cover must be at least 1.5 in.

- The concrete strength must be between 3 and 8 ksi for normal weight concrete and at least 4 ksi for lightweight concrete.
- The maximum yield stress of either structural steel or reinforcing bar must not exceed 60 ksi for calculations.
- For rectangular concrete-filled tubes, the wall thickness for a face with width b must be at least $b\sqrt{F_y / 3E}$.
- The minimum thickness for circular concrete-filled tubes with an outside diameter of D is $D\sqrt{F_y / 8E}$.

The AISC composite column design procedure is similar to the steel column design procedure, except that it uses modified properties calculated from the composite cross-section instead of the steel section properties. A modified yield stress, F_{my}, modified elastic modulus, E_m, and modified radius of gyration, r_m are required to design composite column. These parameters are given by Equations (2-12) and (2-14).

$$F_{my} = F_y + c_1 F_{yr}(A_r / A_s) + c_2 f_c^{'}(A_c / A_s) \tag{2-12}$$

$$E_m = E + c_3 E_c (A_c / A_s) \tag{2-13}$$

$$r_m = \max(r_{steel}, 0.3h_l) \tag{2-14}$$

where,

A_c = area of concrete, in.^{2}

A_r = area of longitudinal reinforcing bars, in.^{2}

A_s = area of steel, in.^{2}

E = modulus of elasticity of steel, ksi

E_c = modulus of elasticity of concrete, ksi

= $w^{1.5}\sqrt{f_c^{'}}$ where w is the unit weight of the concrete in lbs./ft^3 and f'_c is in ksi

F_y = specified minimum yield stress of the steel shape, pipe, or tube, ksi

F_{yr} = specified minimum yield stress of the longitudinal reinforcing bars, ksi

f'_c = specified compressive cylinder strength of concrete, ksi

h_l = overall thickness of entire composite cross-section in the plane of buckling, in.

Depending on the type of composite cross-section, the coefficients c_1, c_2, and c_3 have different values. For concrete filled tubes and pipes these coefficients are $c_1 = 1.0$, $c_2 = 0.85$, and $c_3 = 0.4$; for concrete encased shapes the constants are $c_1 = 0.7$, $c_2 = 0.6$, and $c_3 = 0.2$.

The derivation of the values for these coefficients is not discussed in detail by SSRC Task Group 20 [1979]. The coefficient c_1 in equation 2-12is related to the effectiveness of reinforcement bars. For a CFT cross-section, the outer structural steel confines the internal concrete. Thus, the concrete remains well-confined and spalling is not an issue. The reinforcement bars can reach their full capacity, which means that c_1 is 1.0 for CFTs.

In the case of SRC cross-sections, because the concrete is not as well confined, it is difficult for the bars to achieve their maximum capacity. The American Concrete Institute (ACI) specification has recommended allowing only 70% of the maximum capacity for reinforced concrete column design. Based on this, c_1 is taken as 0.7 for SRC columns.

The coefficient c_2 is related to the concrete part of the cross-section. For the CFT cross-section, the concrete can attain a stress of 0.85 f'_c, because the concrete is confined. Thus, c_2 is 0.85 for CFT's. In the case of SRC cross-sections, ACI recommends to use 70% of f'_c as its capacity for unconfined concrete. This reduction coefficient was product of 0.7 and 0.85 f'_c. The coefficient c_2 was 0.595, so c_2 is 0.6 for SRC columns.

The coefficient c_3 is related to the ratio of the stiffness of the concrete to the stiffness of the overall cross-section. For confined concrete, ACI recommended using only 40% of the stiffness of the concrete, and only 20% could be permitted to use for unconfined concrete. Thus, c_3 is 0.4 for CFT cross-sections and c_3 is 0.2 for SRC cross-sections. As per ACI recommendations, this parameter is related to the influence of cracking and creep.

The maximum modified radius of gyration (r_m) for a solid rectangular section is taken as 30% of its depth. Even though steel and concrete act jointly to resist flexure, the radius of gyration of the steel section is suitable for the whole composite section if the steel with large reinforcement ratios dominates while the radius of gyration of the concrete is suitable if the concrete with low reinforcement ratios dominates the behavior. Thus, the SSRC report selects the larger of the two for use in design.

Once the cross-sectional strength of the composite columns has been determined, the design follows the AISC procedure for steel columns using the modified parameters (modified yield stress, F_{my}, modified elastic modulus, E_m, and modified radius of gyration, r_m). The modified column slenderness parameter, λ_m is defined as

$$\lambda_m = \frac{kl}{r_m \pi}\sqrt{\frac{F_{my}}{E_m}} \tag{2-15}$$

As for steel only sections, based on this slenderness, the critical stress for $\lambda_m \leq 1.5$ is calculated by

$$F_{cr} = (0.658^{\lambda_m})F_{my} \tag{2-16a}$$

and for $\lambda_m > 1.5$ by

$$F_{cr} = \frac{0.877}{\lambda_m{}^2}F_{my} \tag{2-16b}$$

The nominal strength of the column P_n is

$$P_n = A_s F_{cr} \tag{2-17}$$

And design equation is

$$P_u = \phi_c P_n \tag{2-18}$$

The AISC interaction curve for the composite sections is derived by the same method as for steel only beam-columns under combined axial compression and bending moment. The nominal axial strength of the composite section is determined as above. The nominal flexural strength (M_n) can be calculated following traditional approaches for reinforced concrete members or can be taken as:

$$M_n = M_p = ZF_y + 1/3(h_2 - 2c_r)A_r F_{yr} + \left(\frac{h_2}{2} - \frac{A_w F_y}{1.7 f_c' h_l}\right)A_w F_y \tag{2-19}$$

where,

A_w = web area of encased steel shape; for concrete filled tubes, A_w=0, in.^{2}

Z = plastic section modulus of the steel section, in.^{3}

C_r = average distance from compression face to centroid of longitudinal reinforcement in that face and distance from tension face to longitudinal reinforcement in that face, in., = $(C_{rc}+ C_{rt})/2$

C_{rc} = distance from compression face to longitudinal reinforcing steel in that face, in.

C_{rt} = distance from tension face to longitudinal reinforcing steel in that face, in.

h_1 = width of composite cross-section perpendicular to the plane of bending, in.

h_2 = width of composite cross-section parallel to the plane of bending, in.

M_n from Equation (2-19) is an approximation to the bending strength determined from a plastic stress distribution on the composite cross-section. To get the flexural capacity of a composite cross-section, an iterative procedure utilizing strain compatibility principles is required. This equation includes the contributions of three components to the flexural capacity: the structural steel section, the longitudinal reinforcement, and the concrete part. The first part of Equation (2-23) is the plastic bending capacity of the steel which is the product of the plastic section modulus and the yield stress of the steel. The second part of Equation (2-19) assumes that 1/3 of the longitudinal reinforcing bars can be regarded to be concentrated at a position C_r. This is a reasonable and often conservative approximation that considerably simplifies the calculations. The third part of Equation (2-19) reflects the assumption that the web of the steel can be taken as tension reinforcement with its centroid at the location given by the term in parenthesis. For RCFTs or CCFTs, A_w may be taken as the sidewall areas, but A_w equal to 0 is recommended as a conservative as approach by SSRC. Thus, the plastic moment for a CFT is given by the steel tube alone because in general CFTs do not have longitudinal reinforcement.

2.1.3 LRFD Composite Steel Column Design (2005)

The proposed AISC 2005 Unified Specification contains significant changes in the design of composite columns. In this section, these revisions are introduced and compared with the provisions of the 1999 AISC LRFD specification that was discussed in the previous section. Composite column design in the new 2005 Specification is subject to the following limitations:

- The cross sectional area of the steel must comprise at least 1 percent of the composite cross section. This limit was 4% in the 1999 AISC LRFD.
- The SRC section must be reinforced with at least 4 longitudinal continuous the bars steel bars. The number of required bars was not given in the 1999 AISC LRFD, although 4 was the minimum logical choice. However, the new requirement is that these bars must be continuous across the floor slabs.
- Transverse reinforcement must be spaced at least 16 longitudinal bar diameters, 48 tie bar diameters or 0.5 times the least dimension of the composite section. This requirement replaces the extant requirement for spacing given as 2/3 of the least dimension in the 1999 AISC LRDF, and makes the 2005 provisions consistent with those for regular reinforced concrete columns in ACI 318.
- Clear cover must be at least 1.5 in.
- Each area of longitudinal reinforcement and transverse ties must be at least 0.009 in^2. / in. of bar spacing. This represents an increase of 0.002 $in.^2$/in. from the 1999 AISC LRFD.

- The minimum reinforcement ratio for continuous longitudinal reinforcing shall be 0.004. This is a new requirement.
- The concrete strength must be between 3 and 10 ksi for normal weight concrete and between 3 and 6 ksi for lightweight concrete. In the previous specification, the upper limit for normal weigh concrete was 8 ksi. The limits for lightweight have been changed from a minimum of 4 ksi to a range of 3 to 6 ksi from the 1999 AISC LRDF.
- The maximum yield stress of either structural steel or reinforcing bar must not exceed 75 ksi for calculations. This is 25% increase from the 60 ksi allowed in the 1999 AISC LRDF.
- For rectangular concrete-filled tubes, the maximum b/t ratio shall be equal to $2.26\sqrt{E/F_y}$. This has been considerably liberalized from the 1999 AISC LRFD limit of $\sqrt{3\mathrm{E}/\mathrm{F_y}}$.
- The maximum D/t for circular concrete-filled tubes shall be 0.15 E/F_y. This has been liberalized from the 1999 AISC LRDF limit of $\sqrt{8E/F_y}$)

The 2005 AISC Composite column design method has different equations for cross-sectional strength depending on whether columns are encased composite columns and filled composite columns. The cross-sectional strength is based on the plastic capacity of the section.

For encased columns,

$$\alpha = \sqrt{P_o / P_e} \tag{2-20}$$

$$P_0 = A_s F_y + A_{sr} F_{yr} + 0.85 f_c' \tag{2-21}$$

$$P_e = \pi^2 (EI)_{eff} / (KL)^2 \tag{2-22}$$

$$EI_{eff} = E_s I_s + 0.5 E_s I_{sr} + C_1 E_c I_c \tag{2-23}$$

$$C_1 = 0.1 + 2\left(\frac{A_s}{A_c + A_s}\right) \le 0.3 \tag{2-24}$$

For filled composite columns,

$$P_0 = A_s F_y + A_{sr} F_{yr} + C_2 f_c' \tag{2-25}$$

$$EI_{eff} = E_s I_s + 0.5 E_s I_{sr} + C_3 E_c I_c \tag{2-26}$$

$$C_3 = 0.6 + 2\left(\frac{A_s}{A_c + A_s}\right) \le 0.9 \tag{2-27}$$

where

C_2 = 0.85 for rectangular sections and 0.95 for circular sections

A_c = area of concrete, in^2

A_{sr} = area of continuous reinforcing bars, in^2

A_s = area of steel section, in^2

E_c = modulus of elasticity of concrete, ksi

= $w^{1.5}\sqrt{f_c'}$ where w is the unit weight of the concrete in lbs./ft^3 and fc' is in ksi

E_s = modulus of elasticity of steel, shall be taken as 29,000 ksi.

EI_{eff} = effective moment of inertia rigidity of composite section, kip-in.^{2}

f'_c =specified minimum concrete compressive strength, ksi

F_y = yield stress of the steel section, ksi

F_{yr} = specified minimum yield stress of reinforcing bars, ksi

I_c = moment of inertia of the concrete section, in.^{4}

I_s = moment of inertia of the steel shape, in.^{4}

I_{sr} = moment of inertia of reinforcing bars, in.^{4}

K = effective length factor determined in accordance with chapter C

L = laterally unbraced length of the member, in.

w_c = weight of concrete per unit volume

The design strength is given as:

$$\varphi_c P_n \geq P_u \tag{2-28}$$

where the resistance factor is:

$$\phi_c = 0.75 \tag{2-29}$$

and the nominal strength P_n is given by:

$$P_n = \Lambda P_o \tag{2-30}$$

Based on the column slenderness, when $\alpha \leq 1.5$,

$$\Lambda = 0.658^{\alpha^2} \tag{2-31a}$$

and when $\alpha \geq 1.5$ is

$$\Lambda = 0.877/\alpha^2 \tag{2-31b}$$

Equation (2-21), for encased sections, comprises three terms. The first term is related to the structural steel section and the second term is related to the reinforcing bars. Both the structural steel and the reinforcement bars are assumed to reach their full capacity, which means that the coefficients for the first and the second term are 1.0. The third term is related to concrete strength. A uniform compressive stress of 0.85 f'_c is assumed.

Equation (2-23), for the stiffness of the cross section, also has three parts. The structural steel is considered to contribute its full capacity, but the reinforcing bars are considered to contribute only half of their capacity as the bars on the tension side of the section will probably have yielded well before the section attains its ultimate strength. The effectiveness of the concrete part is reduced using the coefficient C_1, because the concrete is not well confined.

The interaction can be taken as shown below, by determining the level of axial load (see Figure 2-1).

For $P_r < P_{cb}$,

And if $\frac{M_{rx}}{M_{cx}} \leq 1$ and $\frac{M_{ry}}{M_{cy}} \leq 1$ then

$$\frac{M_{rx}}{M_{cx}} + \frac{M_{ry}}{M_{cy}} \leq 1 \tag{2-32a}$$

otherwise if $\frac{M_{rx}}{M_{cx}} > 1$ and $\frac{M_{ry}}{M_{cy}} \le 1$ then

$$\frac{P_r}{P_c} + \frac{M_{cbx} - M_{rx}}{M_{cbx} - M_{cx}} + \frac{M_{ry}}{M_{cy}} \le 1 \quad \text{(2-32b)}$$

otherwise if $\frac{M_{rx}}{M_{cx}} \le 1$ and $\frac{M_{ry}}{M_{cy}} > 1$ then

$$\frac{P_r}{P_c} + \frac{M_{rx}}{M_{cx}} + \frac{M_{cby} - M_{ry}}{M_{cby} - M_{cy}} \le 1 \quad \text{(2-32c)}$$

otherwise if $\frac{M_{rx}}{M_{cx}} > 1$ and $\frac{M_{ry}}{M_{cy}} > 1$ then

$$\frac{P_r}{P_c} + \frac{M_{cbx} - M_{rx}}{M_{cbx} - M_{cx}} + \frac{M_{cby} - M_{ry}}{M_{cby} - M_{cy}} \le 1 \quad \text{(2-32d)}$$

If $P_r \ge P_{cb}$,

$$\frac{P_r - P_{cb}}{P_c - P_{cb}} + \frac{M_{rx}}{M_{cx}} + \frac{M_{ry}}{M_{cy}} \le 1 \quad \text{(2-32e)}$$

, where

P_r = required compressive strength, kips

P_c = available compressive strength, kips

= $\phi_b P_n$

P_{cb} = axial compressive strength at balanced moment, kips

M_r = required flexural moment, kip-in.

M_c = available flexural strength, kip-in.

= $\phi_b M_n$

M_b = balanced moment, kip-in.

x = subscript relating symbol to strong axis bending.

y = subscript relating symbol to weak axis bending.

ϕ_c = 0.75

ϕ_b = 0.9

The interaction equations described above result in the interaction surface shown in Figure 2-1.

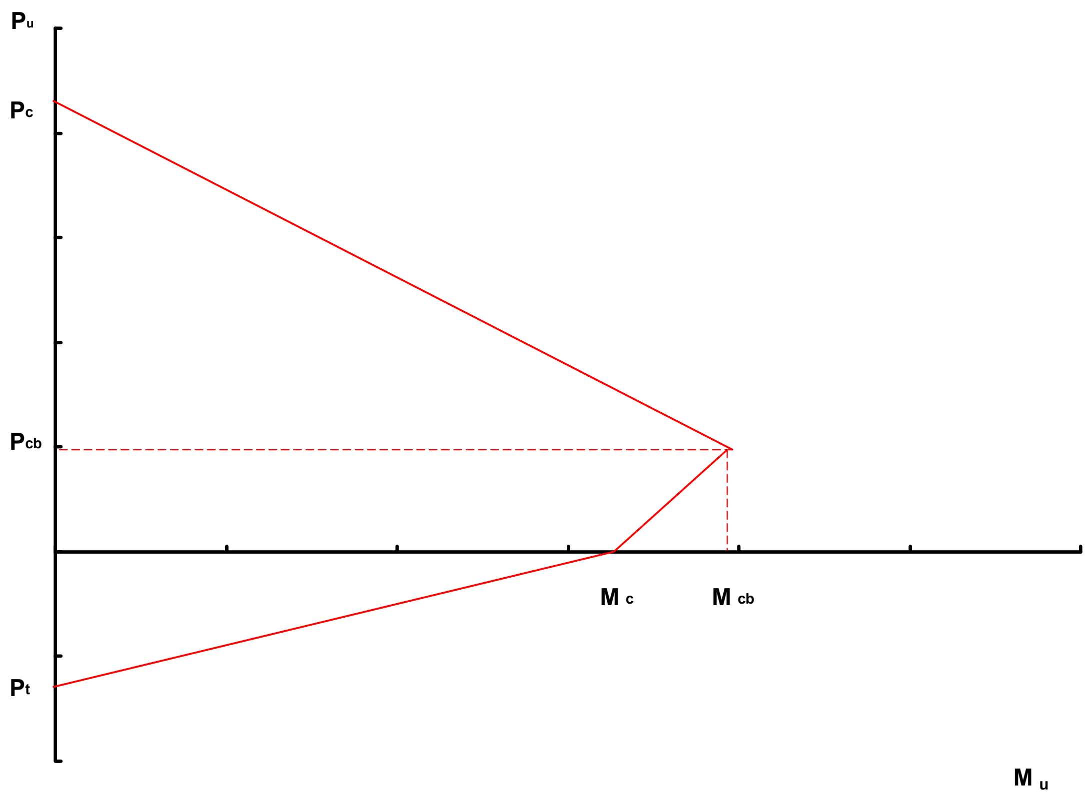


Figure 2-1 Interaction curve for 2005 AISC Specification

2. 2 EUROCODE 4

There are two design methods in the pre-normative version (ENV) of Eurocode 4 (CEN, Eurocode 4: Design of composite steel and concrete structures, European Committee for standardization, 2004). One is a general method which includes columns with non-symmetrical or non uniform cross-section over the column length. The other is a simplified method for columns of doubly symmetric and uniform cross-section over the length. The simplified design method of for compression members for in Eurocode 4 is based on the buckling curves for steel columns which is given by in Eurocode 3 (CEN, Eurocode 3 : Design of steel structures, European Committee for standardization, 2004).

The Eurocode 4 column design assumes that concrete and steel interact fully with each other until failure. Design by the Eurocode method uses the full plastic axial and moment capacity of the cross-section and then reduces those values based on the column slenderness and other factors. The Eurocode composite design considers all material properties of the cross-section, including partial safety factors for the different materials. The Eurocode uses partial safety factors to reduce steel yield stress, concrete compressive strength, and yield stress of reinforcing bar, while AISC uses a single resistance factor. This is one of the reasons why the Eurocode procedures are more complex than the AISC composite column design ones.

2. 2.1 Eurocode 4 Column Design

Before starting to review composite column design by the simplified method, the composite column is required to meet the following limitations:

- The composite column is doubly symmetric and of uniform cross-section over the whole column length.
- The slenderness parameters of the column, λ, is less than 2.0.
- The minimum requirement of the longitudinal reinforcement is 0.3%.
- For encased columns, a minimum cover is of 40 mm is required in order to prevent spalling of the concrete and/or, steel corrosion from due to external environmental factors and, and fire.

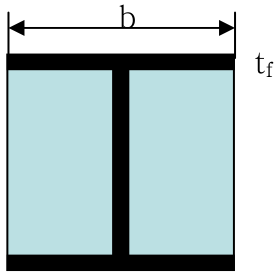
For compression members, local buckling of the steel is checked first. Each type of cross-section must meet certain minimum depth-to-thickness ratios (Table 2.1). For rectangular hollow steel sections, b is the greater overall dimension of the section. For circular hollow steel sections, the diameter is d and the thickness is t. For partially encased I-sections, the flange width is b_f and a flange thickness is t_f.

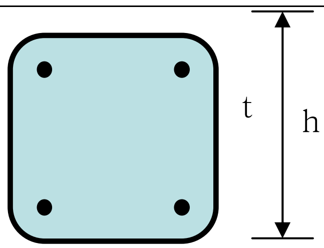
In Table (2-1), the term ε is function of the yield strength of the steel.

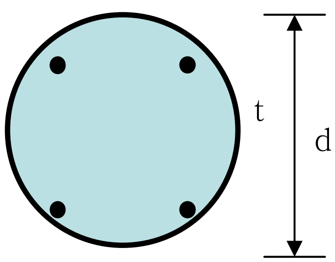
$$\varepsilon = \sqrt{\frac{34.08}{F_y}} \tag{2-33}$$

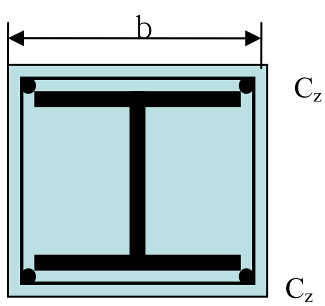
For encased cross-sections, the above calculations can be neglected because local buckling is not likely.

Table 2-1 Limit width to thickness ratio to avoid local buckling

<table>
<tr><td rowspan="2">Type of cross-section</td><td rowspan="2">Limit ratio</td><td colspan="3">Limits for different steel grades</td></tr>
<tr><td>S 235</td><td>S 275</td><td>S 355</td></tr>
<tr><td>

Partially encased I-section</td><td>$b/t_f \leq$</td><td>44ε</td><td>41 ε</td><td>36 ε</td></tr>
<tr><td>

Rectangularhollow steel sections</td><td>$h/t \leq$</td><td>52 ε</td><td>48 ε</td><td>42 ε</td></tr>
<tr><td>

Circular hollow steel section</td><td>$d/t \leq$</td><td>90 ε^2</td><td>77 ε^2</td><td>60 ε^2</td></tr>
<tr><td>

Encased steel section</td><td colspan="4">No check of local buckling for encased cross-section
In order to prevent premature spalling of the concrete, minimum concrete cover may be provided.
$C_z \leq \max imum(40\ mm, b\ /\ 6)$</td></tr>
</table>

The plastic resistance of cross-sections subjected to axial loads is given by P_{pl}. This equation combines the resistance of the structural steel, the concrete and the reinforcement. For encased shapes, the equation is

$$P_{pl} = A_s \frac{F_y}{\gamma_s} + A_c \frac{0.85 f_c'}{\gamma_c} + A_r \frac{F_{yr}}{\gamma_r} \tag{2-34}$$

where A_s, A_c, and A_r are the cross-sectional area of the structural steel, the concrete, and the reinforcement, respectively. For concrete filled rectangular cross-sections, the reduction coefficient of 0.85 in the second term of Equation (2-34) is neglected because the concrete part has a high resistance due to the confinement by the structural steel. The confinement effect is not taken into account when the slenderness ratio of the column λ is greater than 0.5 and the eccentricity of loading, e, is greater than d/10, where d is the outside diameter of the steel tube. Thus, the strength equation for concrete filled circular cross-sections is:

$$P_{pl} = A_s \frac{F_y}{\gamma_s}\eta_2 + A_c \frac{f_c'}{\gamma_c}\left(1 + \eta_1 \frac{t}{d}\frac{F_y}{f_c'}\right) + A_r \frac{F_{yr}}{\gamma_r} \tag{2-35}$$

where

γ_s = partial safety factor for the structural steel = 1.1

γ_c = partial safety factor for the concrete = 1.5

γ_r = partial safety factor for the reinforcing steel = 1.15

The coefficients η_1 and η_2 account for this the confinement effect. The strength of the concrete is increased by η_1 because concrete has a higher strength when a triaxial state of stress occurs. The strength of the steel tube is decreased by η_2 because the effective yield stress of the steel is reduced by the hoop stresses.. Both η_1 and η_2 are related to the slenderness and the eccentricity of the axial load and are defined as

$$\eta_1 = \eta_{10} + (1 - 10\frac{e}{d}) \geq 0.0 \tag{2-36}$$

$$\eta_2 = \eta_{20} + (1 - \eta_{20})10\frac{e}{d} \geq 1.0 \tag{2-37}$$

where

$$\eta_{10} = 4.9 - 18.5\lambda + 17\lambda^2 \geq 0.0 \tag{2-38}$$

$$\eta_{20} = 0.25(3 + 2\lambda) \leq 1.0 \tag{2-39}$$

$$e = \text{eccentricity of loading} = \frac{M_{Sd}}{N_{Sd}} \tag{2-40}$$

where M_{Sd} is the maximum design bending moment calculated by first order theory and N_{Sd} is the design axial load.

The slenderness parameters of the column is defined by

$$\lambda = \sqrt{\frac{A_s F_y + 0.85 A_c f_c' + A_r F_{yr}}{P_E}} \leq 2.0 \tag{2-41}$$

P_E is the Euler buckling load defined as

$$P_E = \frac{(EI)_e \pi^2}{(KL)^2} \tag{2-42}$$

where,

L = buckling length of the column (effective length)

$(EI)_e$ = effective bending stiffness

$$(EI)_e = E_s I_s + E_c I_c + E_r I_r \qquad (2\text{-}43)$$

and

E_s = modulus of elasticity of steel, ksi

E_c = modulus of elasticity of concrete.

E_r = modulus of elasticity of reinforcing steel, ksi.

I_s = moment of inertia of steel, in.^{4}

I_c = moment of inertia of concrete(assumed to be uncracked), in.^{4}

I_r = moment of inertia of reinforcing steel, in.^{4}

The modulus of elasticity for concrete was previously defined in the Eurocode as E_c = 600f'_c but is now defined as $E_c = 0.8E_{cm} / \gamma_c$ where E_{cm} is the secant modulus of concrete, and γ_c is taken as 1.35.

The plastic resistance of composite cross-section, P_{pl}, which is reduced by κ, the buckling reduction factor, must be greater than the design load, P_{Sd}.

$$P_{Sd} \le \kappa P_{pl} \qquad (2\text{-}44)$$

where

κ = reduction factor accounting for the column slenderness

P_{Sd} = design value of the axial force

P_{Pl} = plastic resistance of the cross-section

The buckling reduction factor κ is given in function of λ.

$$\kappa = f_k - \sqrt{f_k^2 - \frac{1}{\lambda^2}} \leq 1.0 \qquad (2\text{-}45)$$

,where

$$f_k = \frac{1 - \alpha(\lambda - 0.2) + \lambda^2}{2\lambda^2} \qquad (2\text{-}46)$$

α = imperfection factor

0.21 for concrete-filled circular and rectangular hollow sections.

0.34 for completely or partly concrete-encased I-section with bending about the major axis of the profile.

0.49 for completely or partly concrete-encased I-section with bending about the minor axis of the profile.

This gives rise to three curves, labeled a, b and c (Figure 2-2). Curve a is for circular and rectangular concrete filled sections. Curve b is for completely or partly concrete-encased I-sections with bending about the major axis of the profile. Finally, curve c is for or partly concrete-encased I-section with bending about the minor axis of the profile. The values for κ are given in Table (2-2). Curve b corresponds to the AISC buckling curve.

Table 2-2 κ values for strut curves a, b and c

λ	**Curve a**	**Curve b**	**Curve c**
0.2	1.0000	1.0000	1.0000
0.3	0.9775	0.9641	0.9491
0.4	0.9528	0.9261	0.8973
0.5	0.9243	0.8842	0.8430
0.6	0.8900	0.8371	0.7854
0.7	0.8477	0.7837	0.7247
0.8	0.7957	0.7245	0.6622
0.9	0.7339	0.6612	0.5998
1.0	0.6656	0.5970	0.5399
1.1	0.5960	0.5352	0.4842
1.2	0.5300	0.4781	0.4338
1.3	0.4703	0.4269	0.3888
1.4	0.4179	0.3817	0.3492
1.5	0.3724	0.3422	0.3145
1.6	0.3332	0.3079	0.2842
1.7	0.2994	0.2781	0.2577
1.8	0.2702	0.2521	0.2345
1.9	0.2449	0.2294	0.2141
2.0	0.2229	0.2095	0.1962

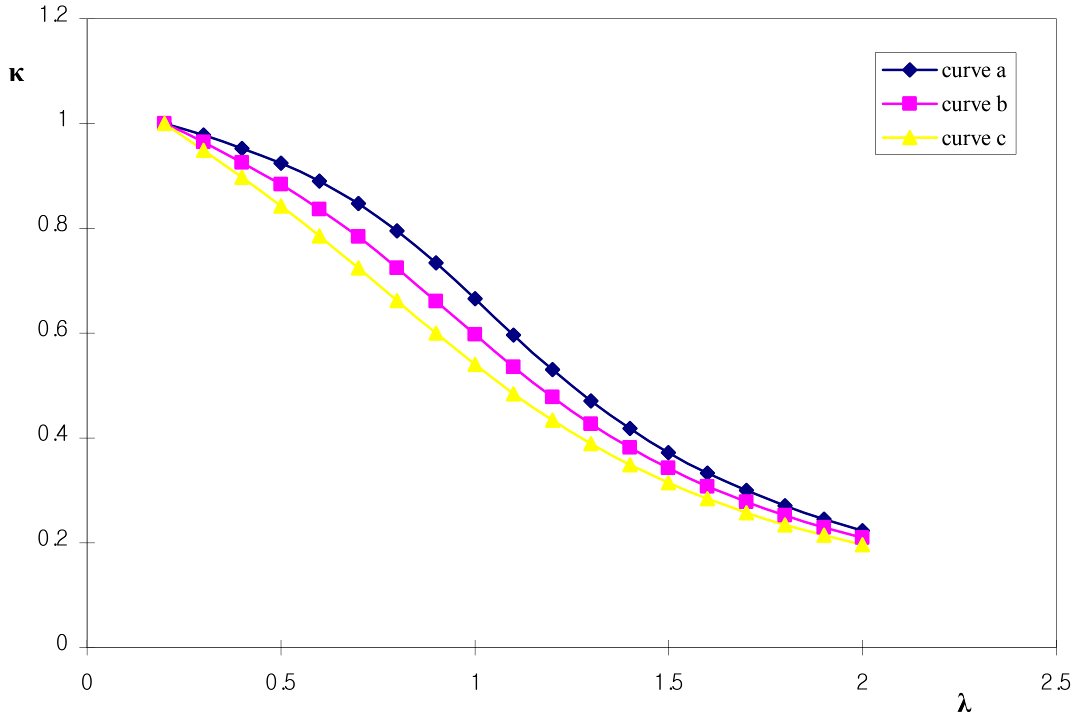


Figure 2-2 European strut curve

2.2.2 Eurocode 4 Beam-Column Design

The resistance of the cross-section subjected to axial load and bending moment can be calculated by a utilizing a full plastic stress distribution assumption. For slender members, second-order effects must be considered. Second-order theory considers the influence of both the local (P-δ) and global (P-Δ) deformations of the column when determining the internal forces and moments.

In the steel beam-column interaction curve, the moment resistance reduces with increasing axial load. However, in the composite beam-column interaction curve, the moment resistance increases up to the "balance point" due to the presence of axial load because of the prestressing effect of the compressive forces. The interaction curve can be drawn by determining the stress block at numerous levels of axial load. This calculation is easily performed by computer routines. An approximation of the full interaction curve can be determined by calculating several points on the curve and connecting those points with straight lines. These points may be calculated by assuming rectangular stress blocks.

2.2.2.1 Approximation by a Polygonal Path

The interaction curve of the cross-section can be approximately drawn by connecting four or five key points related to the resistance to combined compression and bending. Figure 2-3 shows stress distribution at each point from point A to D of the interaction curve for major axis bending. Five points from A to E are required as shown in the stress distribution and interaction curve for minor axis bending (see Figure 2-4). The maximum internal moment at point D is:

$$M_{\max} = Z_s f_{yd} + \frac{1}{2} Z_c f_{cd} + Z_r f_{rd} \tag{2-47}$$

where,

Z_s = plastic modulus of steel cross-section, in.

Z_c = plastic modulus of overall concrete cross-section, in.^{3}

Z_r = plastic modulus of reinforcement, in.^{3}

The 1/2 factor for the concrete term is based on neglecting tension in the concrete. Thus, only half of the cross-section is considered. The plastic modulus of the reinforcement can be expressed as:

$$Z_r = \sum_{i-1}^{n} A_{ri} e_i \tag{2-48}$$

where

A_{ri} = area of one reinforcing bar, in.^{2}

e_i = distance to the bending axis considered, in.

The plastic moment of the composite cross-section resulting from for the region $2h_n$ can be calculated as:

$$M_{pn} = Z_{sn} f_{yd} + \frac{1}{2} Z_{cn} f_{cd} + Z_{rn} f_{rd} \tag{2-49}$$

The sub index n indicates that the only the stresses within $2h_n$ are used for this calculation. The distance h_n and the region $2h_n$ are shown in Table 2-3. The equations about for h_n are different depending on the type of cross-section and the location of the neutral axis. The equations needed to get obtain h_n are given below.

2.2.2.2 Equations for Concrete-Encased I-sections w/bending about Major Axis

The plastic modulus of the steel I-section about its major axis can be obtained from the design tables, or it can be calculated as:

$$Z_s = \frac{(d-2t_f)t_w^2}{4} + b_f t_f (d - t_f) \tag{2-50}$$

The plastic modulus of the concrete is:

$$Z_c = \frac{h_1 h_2}{4} - Z_s - Z_r \tag{2-51}$$

There are three regions to consider for the position of the neutral axis. The procedure for finding the position is iterative. First, assume the distance h_n is located on a particular region, and then calculate h_n by substituting into the appropriate equation (Equations 2-52 to 2-56). If the value for h_n is within the assumed region, the distance h_n has been determined. If not, another region is chosen and the procedure repeated. The distance h_n and plastic modulus of the steel at for each position are:

(a) Neutral Axis in the web : $h_n \le d/2 - t_f$

$$h_n = \frac{N_{pm} - A_{rn}(2f_{rd} - f_{cd})}{2h_l f_{cd} + 2t_w(2f_{yd} - f_{cd})} \tag{2-52}$$

$$Z_{sn} = t_w h_n^2 \tag{2-53}$$

(b) Neutral Axis in flange : $d/2 - t_f \le h_n < d/2$

$$h_n = \frac{N_{pm} - A_{rn}(2f_{rd} - f_{cd}) + (b_f - t_w)(d - 2t_f)(2f_{yd} - f_{cd})}{2h_l f_{cd} + 2b_f(2f_{yd} - f_{cd})} \tag{2-54}$$

$$Z_{sn} = b_f h_n^2 - \frac{(b_f - t_w)(d - 2t_f)^2}{4} \tag{2-55}$$

(c) Neutral Axis outside the steel section : $d/2 \le h_n \le h_2/2$

$$h_n = \frac{N_{pm} - A_{rn}(2f_{rd} - f_{cd}) - A_s(2f_{yd} - f_{cd})}{2h_l f_{cd}} \tag{2-56}$$

$$Z_{sn} = Z_s \tag{2-57}$$

where A_{rn} is the sum of reinforcement areas within the 2 h_n region, and the plastic modulus of the concrete in the region 2 h_n, is

$$Z_{cn} = h_1 h_n^2 - Z_{sn} - Z_{rn} \tag{2-58}$$

The neutral axis is in the web for most I- type composite sections under major axis bending.

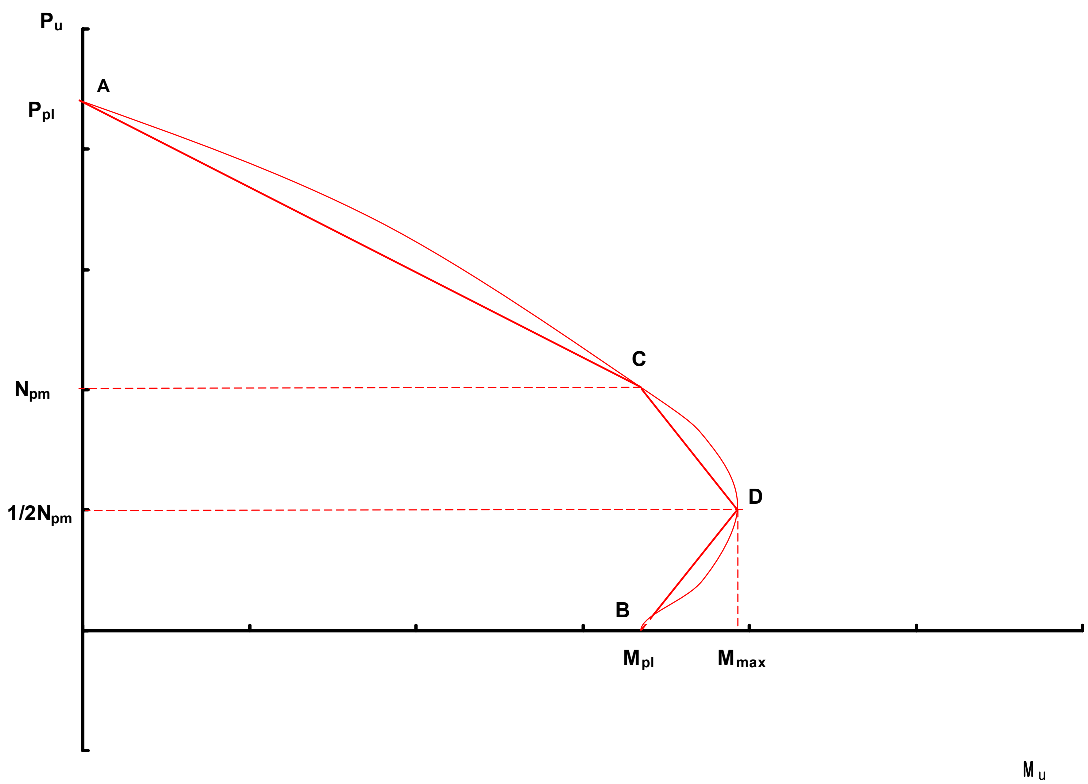
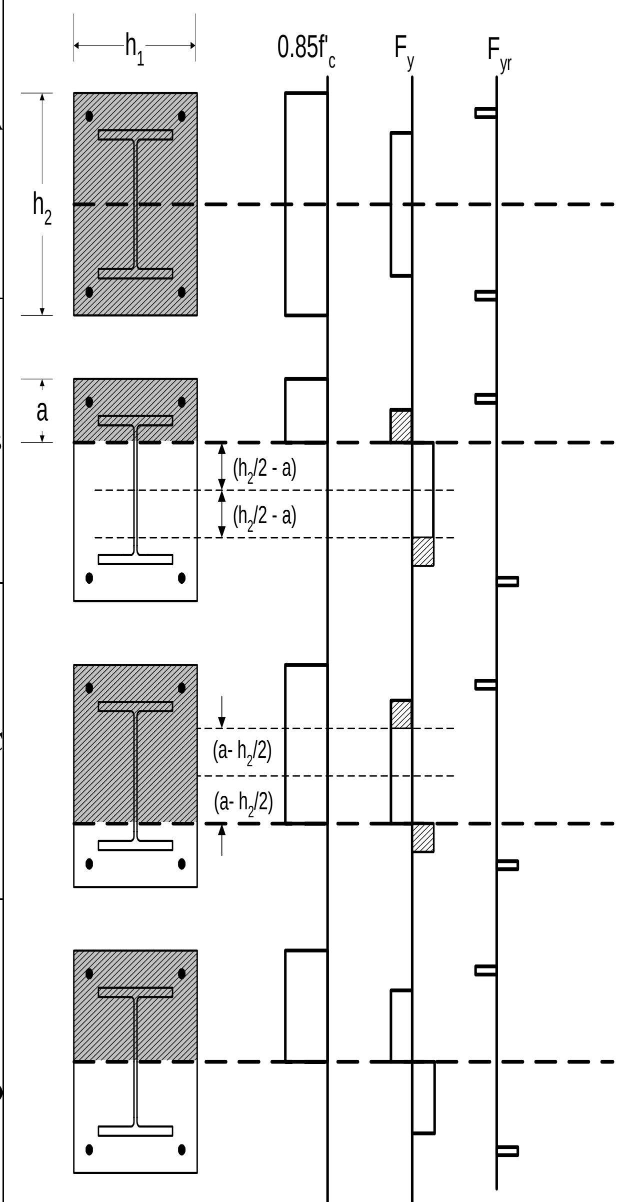


Figure 2-3 SRC major axis bending

Table 2-3 SRC stress distribution at each point (SRC major axis bending)



	Section	Stress Distribution	Equation
A			$N = P_{pl} = A_c \cdot f_{cd} + A_s \cdot f_{yd} + A_r \cdot f_{rd}$ $f_{cd} = 0.85 \cdot f'_c / \gamma_c\ ,\ f_{yd} = F_y / \gamma_s\ ,\ f_{rd} = F_{yr} / \gamma_r$ $\gamma_c, \gamma_s, \gamma_r$: partial safety factors $A_c = h_1 \cdot h_2 - A_s - A_r$ $M = 0$
B			$N = 0$ $h_n \rightarrow h_1 \cdot a \cdot f_{cd} = (h_2 - 2a) \cdot t_w \cdot f_{yd}$ $M = M_{pn} = Z_{sn} \cdot f_{yd} + \frac{1}{2} \cdot Z_{cn} \cdot f_{cd} + Z_{rn} \cdot f_{rd}$ $Z_{sn} = t_w \cdot h_n^2\ ;\ Z_{cn} = h_1 \cdot h_n^2 - Z_{sn} - Z_{rn}$
C			$N = N_{pm} = A_c \cdot f_{cd}$ $h_n \rightarrow h_1 \cdot a \cdot f_{cd} = (h_2 - 2a) \cdot t_w \cdot f_{yd}$ $M = M_{pn} = Z_{sn} \cdot f_{yd} + \frac{1}{2} \cdot Z_{cn} \cdot f_{cd} + Z_r \cdot f_{rd}$
D			$N = \frac{1}{2} \cdot N_{pm} = \frac{1}{2} \cdot A_c \cdot f_{cd}$ $M = M_{max} = Z_s \cdot f_{yd} + \frac{1}{2} \cdot Z_c \cdot f_{cd} + Z_r \cdot f_{rd}$ $Z_s = \frac{(d - 2 \cdot t_f) \cdot t_w^2}{4} + b_f \cdot t_f \cdot (d - t_f)$ $Z_c = \frac{h_1 \cdot h_2^2}{4} - Z_s - Z_r$

2.2.2.3 Equations for encased I-shapes w/ Bending about the Minor Axis

The plastic modulus of the steel I-section about its minor axis can be taken from a table or calculated as:

$$Z_s = \frac{(d - 2t_f)t_w^2}{4} + \frac{2t_f b_f^2}{4} \tag{2-59}$$

The plastic modulus of the concrete is obtained given by:

$$Z_c = \frac{h_1 h_2}{4} - Z_s - Z_r \tag{2-60}$$

There are two regions to consider for the location of the neutral axis for minor axis bending. The same procedure is followed to find the location of the neutral axis as for major axis bending.

(a) Stress Neutral Axis in the flanges $t_w / 2 \le h_n \le b_f / 2$

$$h_n = \frac{N_{pm} - A_{rn}(2f_{rd} - f_{cd}) - t_w(2t_f - d)(2f_{yd} - f_{cd})}{2h_l f_{cd} + 4t_f(2f_{yd} - f_{cd})} \tag{2-61}$$

$$Z_{sn} = 2t_f h_n^2 + \frac{(d - 2t_f)t_w^2}{4} \tag{2-62}$$

(b) Stress Neutral Axis in the flanges $b_f / 2 \le h_n \le h_2 / 2$

$$h_n = \frac{N_{pm} - A_{rn}(2f_{rd} - f_{cd}) - A_s(2f_{yd} - f_{cd})}{2h_l f_{cd}} \tag{2-63}$$

$$Z_s = Z_s \tag{2-64}$$

$$Z_{cn} = h_1 h_n^2 - Z_{sn} - Z_{rn} \tag{2-65}$$

Because the interaction diagram for weak axis bending bulges significantly between Points A`and C, an additional point E is calculated in the region between point A and C

(Figure 2-4). This position can be calculated by arbitrarily choosing a neutral axis between h_n and the edge of the cross-section. It is convenient to choose the edge of the steel shape when making this choice. The result of the axial force calculation at point E is

$$N_E = h_2(h_E - h_n) f_{cd} + 2t_f (h_E - h_n)(2 f_{yd} - f_{cd}) + A_{rE}(2 f_{rd} - f_{cd}) + N_{pm} \qquad (2\text{-}66)$$

where

,A_{rE} = reinforcement area which eventually exists in the additionally compressed region between the distances h_n and h_E.

Finally, the moment M_E is obtained from the difference between M_{max} and ΔM_E.

$$M_E = M_{\max} - \Delta M_E \qquad (2\text{-}67)$$

where

$$\Delta M_E = Z_{sE} f_{yd} + \frac{1}{2} Z_{cE} f_{cd} + Z_{rE} f_{rd} \qquad (2\text{-}68)$$

The terms Z_{sE} , Z_{cE} ,and Z_{rE} can be calculated from the appropriate above equations by substituting h_E instead of h_n.

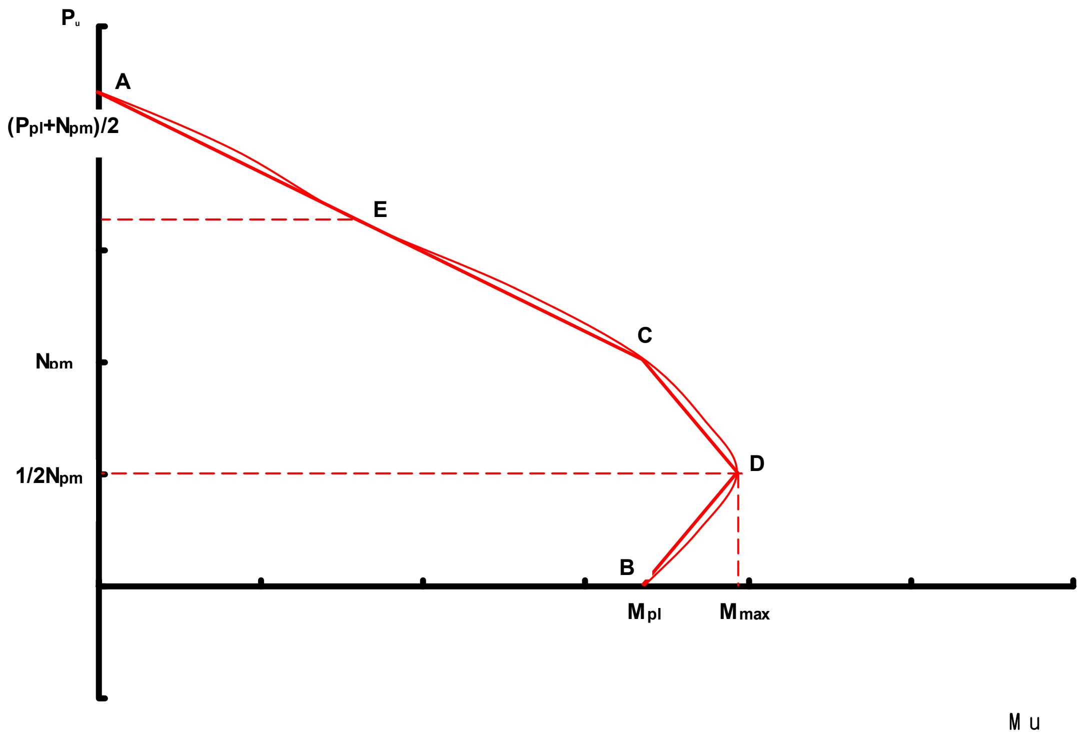
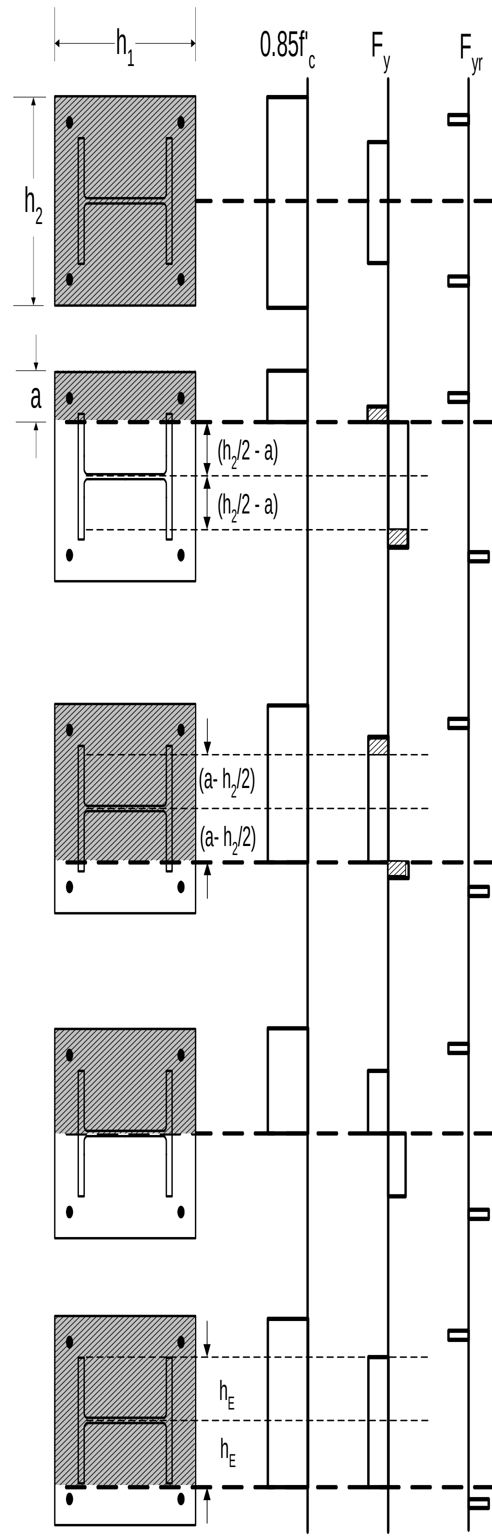


Figure 2-4 SRC minor axis bending

Table 2-4 Stress distribution at each point (SRC minor axis bending)



	Section / Stress Distribution	Equation
A		$N = P_{pl} = A_c \cdot f_{cd} + A_s \cdot f_{yd} + A_r \cdot f_{rd}$ $A_c = h_1 \cdot h_2 - A_s - A_r$ $f_{cd} = 0.85 \cdot f'_c / \gamma_c$, $f_{yd} = F_y / \gamma_s$, $f_{rd} = F_{yr} / \gamma_r$ $M = 0$
B		$N = 0$ $h_n \rightarrow h_1 \cdot a \cdot f_{cd} = 2 \cdot (h_2 - 2a) \cdot t_f \cdot f_{yd} + (d - 2 \cdot t_f) \cdot t_w \cdot f_{yd}$ $M = M_{pn} = Z_{sn} \cdot f_{yd} + \frac{1}{2} \cdot Z_{cn} \cdot f_{cd} + Z_{rn} \cdot f_{rd}$ $Z_{sn} = 2 \cdot t_f \cdot h_n^2 + \frac{(d - 2 \cdot t_f) \cdot t_w^2}{4}$ $Z_{cn} = h_1 \cdot h_n^2 - Z_{sn} - Z_{rn}$
C		$N = N_{pm} = A_c \cdot f_{cd}$ $M = M_{pn} = Z_{sn} \cdot f_{yd} + \frac{1}{2} \cdot Z_{cn} \cdot f_{cd} + Z_r \cdot f_{rd}$
D		$N = \frac{1}{2} \cdot N_{pm} = \frac{1}{2} \cdot A_c \cdot f_{cd}$ $M = M_{max} = Z_s \cdot f_{yd} + \frac{1}{2} \cdot Z_c \cdot f_{cd} + Z_r \cdot f_{rd}$ $Z_s = \frac{(d - 2 \cdot t_f) \cdot t_w^2}{4} + \frac{1}{2} \cdot t_f \cdot b_f^2$ $Z_c = \frac{h_1 \cdot h_2^2}{4} - Z_s - Z_r$
E		$N = \frac{1}{2} \cdot (P_{pl} + N_{pm})$ $M = M_{max} - \Delta M_E, \Delta M_E = Z_{sE} \cdot f_{yd} + \frac{1}{2} \cdot Z_{cE} \cdot f_{cd} + Z_{rE} \cdot f_{rd}$ $Z_{sE} = 2 \cdot t_f \cdot h_E^2 + \frac{(d - 2 \cdot t_f) \cdot t_w^2}{4}$ $Z_{cE} = h_1 \cdot h_E^2 - Z_{sE} - Z_{rE}$

2.2.2.4 Equations for circular and rectangular concrete-filled tubes

For rectangular tubes with dimensions b and h:

$$Z_c = \frac{(b-2t)(h-2t)^2}{4} - \frac{2}{3}r^3 - r^2(4-\pi)(\frac{h}{2} - t - r) - Z_r \quad (2\text{-}69)$$

where Z_r is zero for the most cases because reinforcement bars are generally not required for concrete filled cross-sections. If reinforcement is present, then Equation 2-70 is used. For the steel tube, the plastic modulus of the steel can be taken from a table or calculated by

$$Z_s = \frac{bh^2}{4} - \frac{2}{3}(r+t)^3 - (r+t)^2(4-\pi)(\frac{h}{2} - t - r) - Z_c - Z_r \quad (2\text{-}70)$$

where r, the corner radius in a rectangular tube, can be assumed to be zero because r is not a large number.

Then

$$h_n = \frac{N_{pm} - A_{rn}(2f_{rd} - f_{cd})}{2bf_{cd} + 4t(2f_y - f_{cd})} \quad (2\text{-}71)$$

Also,

$$Z_{cn} = (b-2t)h_n^2 - Z_{rn} \quad (2\text{-}72)$$

$$Z_{sn} = bh_n^2 - Z_{cn} - Z_{rn} \quad (2\text{-}73)$$

For concrete filled cross-sections, the point E is also required. h_E is assumed as half of the distance between h_n and the outside of the section.

$$h_E = h_n/2 + h/4 \quad (2\text{-}74)$$

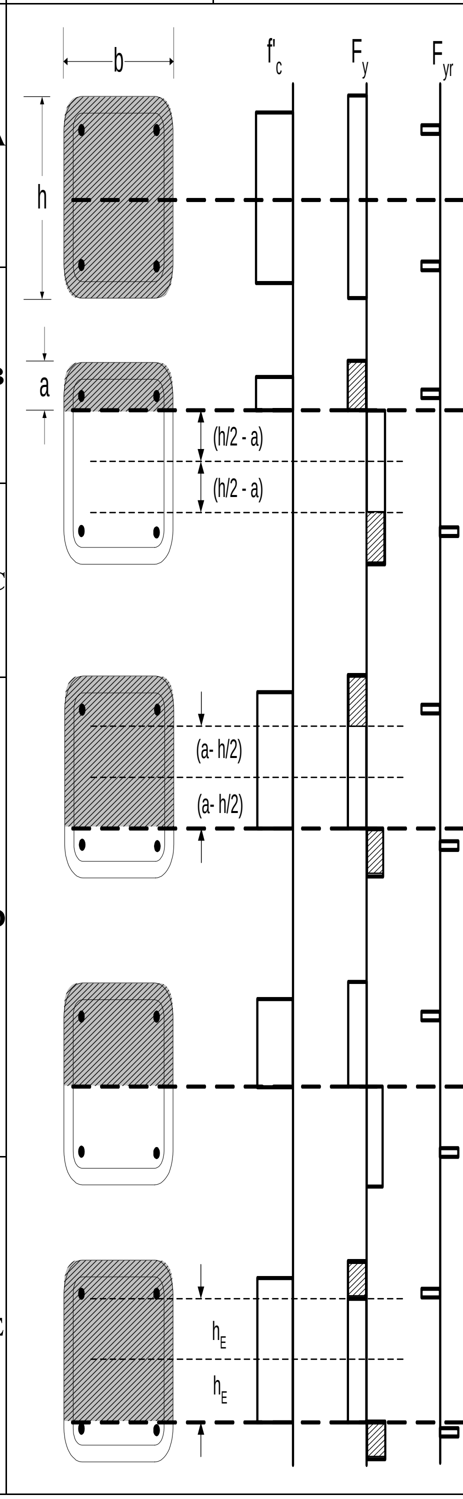
Then

$$N_E = b(h_E - h_n) f_{cd} + 2t(h_E - h_n)(2 f_{yd} - f_{cd}) + A_{rE}(2 f_{rd} - f_{cd}) + N_{pm} \qquad \text{(2-75)}$$

The moment M_E is then determined by equation Equations (2-67) and (2-68). The above equations can be modified for circular cross-sections by substituting h and b with d and r with ((d/2)-t).

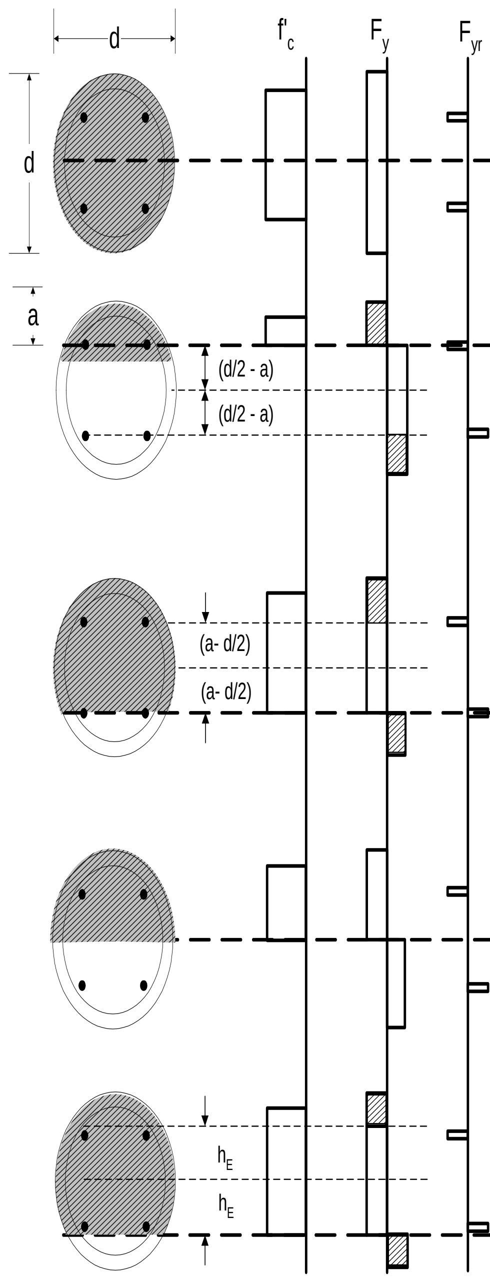
Finally, the moment M_E for concrete filled cross-sections is obtained from the difference between M_{max} and ΔM_E.

Table 2-5 Stress distribution (RCFT cross-section)



	Section	Stress Distribution	Equation
A			$N = P_{pl} = A_s \cdot f_{yd} \cdot \eta_2 + A_c \cdot f_{cd} \cdot \left(1 + \eta_1 \cdot \frac{t}{h} \cdot \frac{F_y}{f'_c}\right) + A_r \cdot f_{rd}$ $A_c = h_1 \cdot h_2 - A_s - A_r$ $f_{cd} = f'_c / \gamma_c$; η_1, η_2 : concrete confinement factors $M = 0$
B			$N = 0$; $M = M_{pn} = Z_{sn} \cdot f_{yd} + \frac{1}{2} \cdot Z_{cn} \cdot f_{cd} + Z_{rn} \cdot f_{rd}$ $Z_{cn} = (b - 2 \cdot t) \cdot {h_n}^2 - Z_{rn}$ $Z_{sn} = b \cdot {h_n}^2 - Z_{cn} - Z_{rn}$
C			$N = N_{pm} = A_c \cdot f_{cd}$ $M = M_{pn} = Z_{sn} \cdot f_{yd} + \frac{1}{2} \cdot Z_{cn} \cdot f_{cd} + Z_r \cdot f_{rd}$
D			$N = \frac{1}{2} \cdot N_{pm} = \frac{1}{2} \cdot A_c \cdot f_{cd}$ $M = M_{max} = Z_s \cdot f_{yd} + \frac{1}{2} \cdot Z_c \cdot f_{cd} + Z_r \cdot f_{rd}$ $Z_c = \frac{(b - 2 \cdot t) \cdot (h - 2 \cdot t)^2}{4} - \frac{2}{3} \cdot r^3 - r^2 \cdot (4 - \pi) \cdot \left(\frac{h}{2} - t - r\right) - Z_r$ $Z_s = \frac{b \cdot h^2}{4} - \frac{2}{3} \cdot (r + t)^3 - (r + t)^2 \cdot (4 - \pi) \cdot \left(\frac{h}{2} - t - r\right) - Z_c - Z_r$
E			$N = \frac{1}{2} \cdot (P_{pl} + N_{pm})$ $M = M_{max} - \Delta M_E, \Delta M_E = Z_{sE} \cdot f_{yd} + \frac{1}{2} \cdot Z_{cE} \cdot f_{cd} + Z_{rE} \cdot f_{rd}$ $h_E = \frac{h_n}{2} + \frac{h}{4}$ $Z_{cE} = (b - 2 \cdot t) \cdot {h_E}^2 - Z_{rE}$; $Z_{sE} = b \cdot {h_E}^2 - Z_{cE} - Z_{rE}$

Table 2-6 Stress distribution at each point (CCFT cross-section)

	Section	Stress Distribution	Equation
A			$N = P_{pl} = A_s \cdot f_{yd} \cdot \eta_2 + A_c \cdot f_{cd} \cdot \left(1 + \eta_1 \cdot \frac{t}{d} \cdot \frac{F_y}{f'_c}\right) + A_r \cdot f_{rd}$ $A_c = \pi \cdot \left(\frac{d}{2}\right)^2 - A_s - A_r$ $f_{cd} = f'_c / \gamma_c$; η_1, η_2 : concrete confinement factors ; $M = 0$
B			$N = 0$; $M = M_{pn} = Z_{sn} \cdot f_{yd} + \frac{1}{2} \cdot Z_{cn} \cdot f_{cd} + Z_{rn} \cdot f_{rd}$ $Z_{cn} = (d - 2 \cdot t) \cdot {h_n}^2 - Z_{rn}$; $Z_{sn} = d \cdot {h_n}^2 - Z_{cn} - Z_{rn}$
C			$N = N_{pm} = A_c \cdot f_{cd}$ $M = M_{pn} = Z_{sn} \cdot f_{yd} + \frac{1}{2} \cdot Z_{cn} \cdot f_{cd} + Z_r \cdot f_{rd}$
D			$N = \frac{1}{2} \cdot N_{pm} = \frac{1}{2} \cdot A_c \cdot f_{cd}$ $M = M_{max} = Z_s \cdot f_{yd} + \frac{1}{2} \cdot Z_c \cdot f_{cd} + Z_r \cdot f_{rd}$ $Z_c = \frac{(d - 2 \cdot t) \cdot (d - 2 \cdot t)^2}{4} - \frac{2}{3} \cdot \left(\frac{d}{2} - t\right)^3 - Z_r$ $Z_s = \frac{b \cdot h^2}{4} - \frac{2}{3} \cdot \left(\frac{d}{2}\right)^3 - Z_c - Z_r$
E			$N = \frac{1}{2} \cdot (P_{pl} + N_{pm})$ $M = M_{max} - \Delta M_E, \Delta M_E = Z_{sE} \cdot f_{yd} + \frac{1}{2} \cdot Z_{cE} \cdot f_{cd} + Z_{rE} \cdot f_{rd}$ $h_E = \frac{h_n}{2} + \frac{d}{4}$; $Z_{cE} = (d - 2 \cdot t) \cdot {h_E}^2 - Z_{rE}$; $Z_{sE} = d \cdot {h_E}^2 - Z_{cE} - Z_{rE}$

K_e = a correction factor that should be taken as 0.6.

E_{cm} is the secant modulus of concrete.

Basically, Equation (2-86) is same as Equation (2-43), with a correction factor K_e used instead of $0.8/\gamma_c$.

Finally, the plastic resistance of composite cross-section is calculated by Equation (2-44).

2.2.2.5 Axial Compression and Uniaxial Bending

The resistance of the composite column under axial compression and uniaxial bending is calculated by using the normalized interaction curve (Figure 2-5). The value κ is plotted on the y-axis of the interaction curve and μ_k is found. This κ is the ratio of M_{pl} to the moment at minimum eccentricity, and is plotted along the x-axis of the interaction curve. A portion of the term μ_k cannot be utilized to resist external loads because any moment due to geometric imperfections is considered in the the beam-column. The value μ_k corresponds to a linear variation of κ to. κ_n, which reflects changes in the moment along the length (analogous to C_b in AISC)

$$\kappa_n = \kappa \frac{(1-r)}{4} \qquad -1 \le r \le 1 \qquad (2\text{-}76)$$

r = ratio of the smaller end moment to the greater moment

This equation implies a larger reduction for columns subjected to a uniform moment distribution along the length and a smaller reduction for columns subjected to double curvature. Imperfection effects are neglected for axial forces less than κ_n. In some cases, μ, may be greater than 1.0. If the axial force and moment are related to each other, the value, μ is should be limited 1.0.

The design equation is then

$$M_{Sd} \le 0.9 \mu M_{pl} \qquad (2\text{-}77)$$

where M_{Sd} is the design moment and

$$\mu = \mu_d - \mu_k \tag{2-78}$$

The factor 0.9 accounts for analysis approximations. The design moments M_{Sd} should include both first and second order effects. First order moments as the design moments can be determined, if slenderness is satisfied by Equation (2-79).

$$\lambda \le 0.2(2 - r) \tag{2-79}$$

If the slenderness limit by Equation (2-79) is not satisfied, the bending moment by second order theory is calculated by the product of a factor k and moment by first order theory.

$$\text{M (Second order effects)} = \text{k M (First order effects)} \tag{2-80}$$

The factor k is defined as

$$k = \frac{\beta}{(1 - \frac{P}{P_{cr}})} \ge 1.0 \tag{2-81}$$

where β is dependent on the moment distribution in the member. β can be taken as 1.0, when moment distribution is caused by lateral loads in isolated columns. When end moments cause the bending, β is defined as

$$\beta = 0.66 + 0.44\frac{M_1}{M_2} \ge 0.44 \tag{2-82}$$

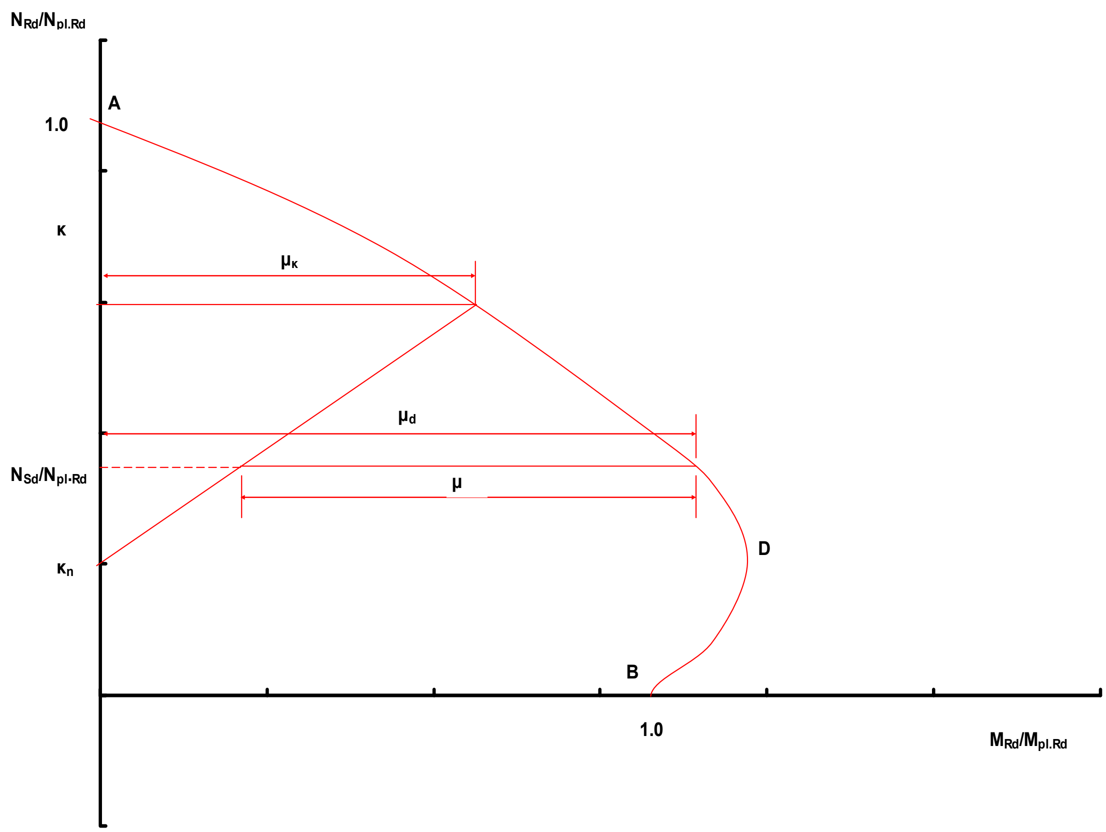


Figure 2-5 Design values for Eurocode beam-columns

2.2.2.6 Compression and Biaxial bending

Both values along the principal axes and a value of μ should be calculated for both curves on the interaction curve. Thus, μ_y and μ_z are used for plotting the interaction curve. Imperfections should be considered for bending in the plane where failure occurs. The interaction equation for bending about the y-axis and the z-axis is:

$$\frac{M_{y.Sd}}{M_{pl.y}\mu_y} + \frac{M_{z.Sd}}{M_{pl.z}\mu_z} \leq 1.0 \qquad (2\text{-}83)$$

Neither of the terms should be greater than 0.9.

2.2.3 New Eurocode 4

The final version of the Eurocode (ENV 1994-1-1: 2004) contains some significant changes to the ENV version. The prenormative version, which was described in the previous section, was issued in 1997 for trial use and comment. A final version, labeled the EV one, was recently completed and will be issued as soon as the translation into all languages is available.

In the EV version, the plastic resistance of cross-section combines all resistance of the structural steel, the concrete, and the reinforcement. The partial safety factors are now embedded within the material strength definitions. The new equation, which is equivalent to Equation (2-34) is:

$$P_{pl} = A_s F_y + 0.85 A_c f_c^{'} + A_r F_{yr} \tag{2-84}$$

Equation (2-84) is used for encased cross-section. For rectangular concrete filled cross-section, Equation (2-84) is also used, but the coefficient 0.85 has been changed to 1.0. For circular concrete filled cross-section, the equation for plastic resistance is

$$P_{pl} = A_s F_y \eta_2 + A_c f_c^{'} (1 + \eta_1 \frac{t}{d} \frac{F_y}{f_c'}) + A_r F_{yr} \tag{2-85}$$

All parameters are explained in Equations (2-36) to (2-46). However, the definition for EI has been changed to;

$$(EI)_e = E_s I_s + K_e E_{cm} I_c + E_r I_r \tag{2-86}$$

Chapter Ⅲ

DATABASE DEVELOPMENT

3.1 Background

Initially, the database was populated with as many tests on composite columns and beam-columns as could be found in the open literature. No effort was made to limit the database to tests which complied with the material and geometric limitations present in current specifications. In addition, both tests subjected to cyclic and monotonic load and with single and double curvature were included. However, only those with monotonic loading and single curvature were used in the comparisons.

The composite column data was composed primarily of three cross-section types: encased shapes or steel-concrete columns (SCR), circular concrete filled tubes (CCFT) and rectangular concrete filled tubes (RCFT). Each category had columns subjected to purely axial load and beam-columns subjected to eccentric load.

Once the test data had been collected, the material and section properties were checked. The material properties collected consisted of the concrete compressive strength, the yield stress of the steel section and reinforcing bars, and the modulus of elasticity of the steel and concrete. Sectional properties included the area, moments of inertia, elastic and plastic section modulus, and radius of gyration of the steel section, reinforcing bars and concrete. In addition both the effective length and the structural steel ratio were recorded. For encased columns, both the longitudinal and transverse reinforcement were included. For circular concrete filled tube columns, the diameter, thickness and diameter-to-thickness ratio were added to the database. Similarly, for rectangular concrete filled tube columns, the depth, thickness, and depth-to-thickness ratio were added. Finally, the experimental value for the ultimate axial load was included for columns and any load eccentricity and ultimate moment reported were added for beam-columns.

Once the main properties had been established, the axial strength for each specimen, as calculated by the AISC and Eurocode methods, were computed. The AISC 1999 specification (see Section I.2) utilizes a modified yield stress, F_{my}, a modified elastic modulus, E_m, and modified radius of gyration, r_m. These quantities were added to the database, along with the capacity predicted by this procedure. For beam-columns, the interaction equations were solved for the case of a constant eccentricity, i.e. neglecting any second-order effects. The latter could not be found in the reports or inferred for many of the tests. This procedure gave Point I in Figure 3-1. In these all calculations, unfactored loads were used. For circular and rectangular concrete filled sections, local buckling checks were also carried out. Finally, the ratio of experimental-to-predicted axial load was calculated.

For the AISC 2005 specification, the plastic resistance to axial load, P_o, the buckling load, P_n, and the effective rigidity (EI_{equiv}) of the composite cross-section were computed. For beam-columns, a predicted load and moment were calculated from the interaction curve. This interaction diagram was drawn using a simplified method which utilizes key points from A to E (Figure 3-1). A slenderness reduction based on the parameter, λ_c was then applied, and the axial capacity for column reduced from point i to point i_λ (i=A…E). Finally, for comparison to design values, the resistance factor, ϕ_c, was applied, and axial capacity reduced from i_λ to i_d. For design comparison, the flexural capacity of the section was also reduced by resistance factor, ϕ_b, from point i to point ϕ_bi. For design comparisons, the test value of test-to-prediction is given as the ratio of OI to OG. For unfactored comparisons, the value is given as the ratio of OI to OH. Exact expressions for determining points A through E are given in the next chapter.

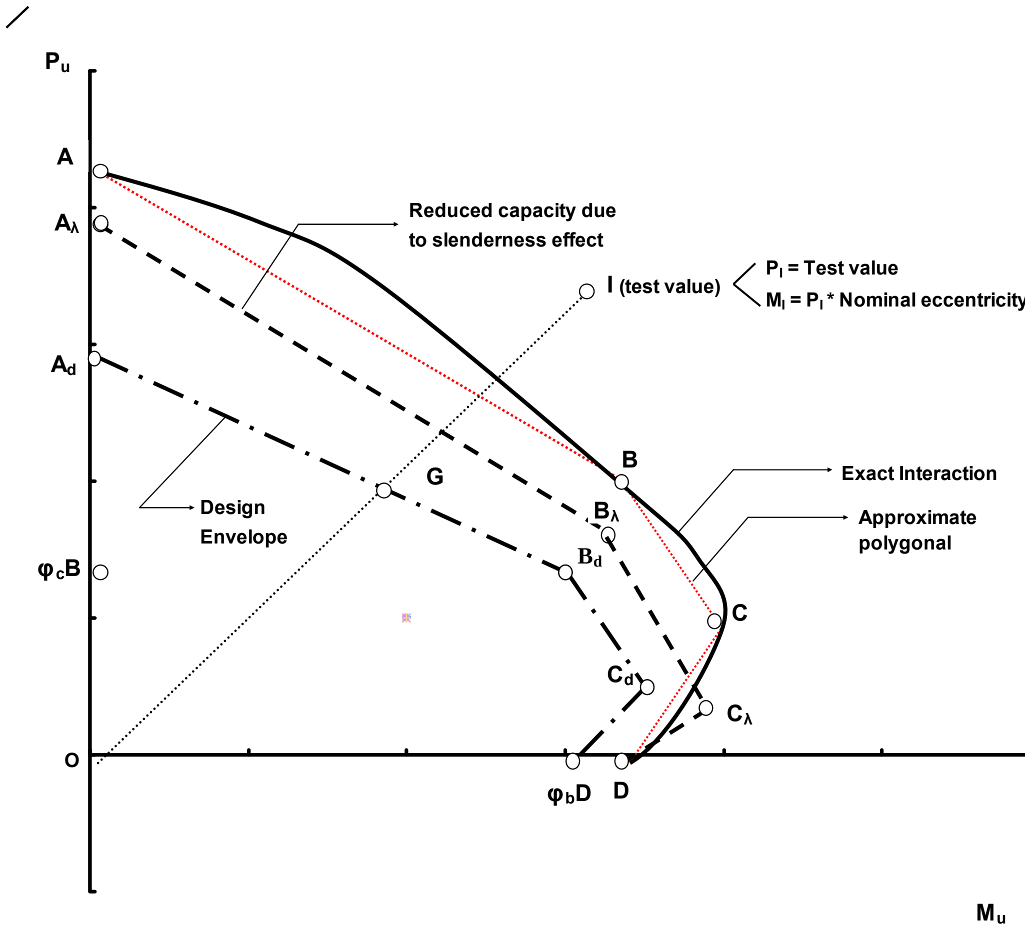


Figure 3-1 AISC interaction diagram

The Eurocode data included the plastic resistance to axial load, N_{pl}, the buckling load, N_{cr}, and the moment of inertia of the composite section. For beam-columns, the predicted load and moment was calculated by interpolation between the simplified points in the interaction diagram. The interaction curve was calculated by full plastic theory and approximated by a polygonal path (Figure 3-2). Point A is plastic capacity of the column. Point B is characterized by the condition that there is no net axial force. At Point C, the resistance of the moment is the same magnitude as point B, but the resistance to axial force is taken as that of the concrete portion only. Point D corresponds to the maximum moment because the neutral axial lies in the center of the cross-section (for doubly symmetric sections). For each point, the resistance of the cross-section, M_{pl}, was calculated.

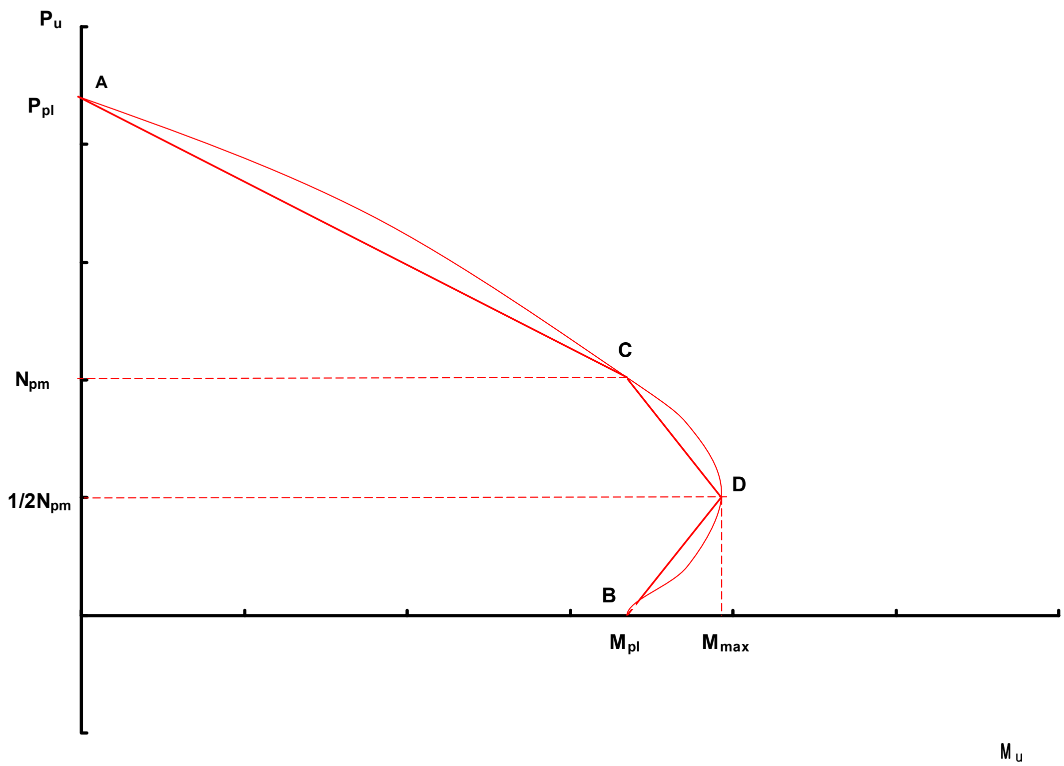


Figure 3-2 Eurocode interaction diagram

3.2 Review of Test Data Added

The following section gives a short summary of the test series that were added to the database. The number given to each test in the database is based on the order in which it appears in this section. The specimen names used were obtained from the original reports. For each data set, there is a brief description that includes the following items :

- •General. Cross-section dimensions, steel section used, and amount of reinforcement, if any.
- •Steel. Yield stress of both structural steel and reinforcing steel.
  Structural steel ratio of composite cross section.
- •Concrete. Typical compressive strength of concrete used, tested as cube or cylinder.
- •Length. The effective length, boundary conditions.

Whenever possible, the units were converted to U.S units.

3.2.1 SRC Columns

<u>SRC columns 82-87</u>

General. As part of a test series that comprised both columns and beam-columns, Han et al. (1992) tested 6 SRC columns. The test parameters varied included the cross-section, ratio of structural steel to gross area of cross section and ratio of reinforcing hoops to gross area of cross-section. The depth of the columns was 6.3 in. (160 mm). The section size of the column was manufactured at 1/3 scale of typical columns generally used in real world applications. The ratio of structural steel to gross area of cross section ranged from 3.8 % to 8.6 %.

Steel. Three tensile coupons were cut from the flange and web of each steel section, reinforcing bars, and hoops in accordance with Korean standard B 0801. The yield stress of the flange ranged from 42.5 ksi (2.9 t/cm^2) to 43.2 ksi (3.0 t/cm^2), and the yield stress of the web ranged from 33.6 ksi (2.3 t/cm^2) to 44.8 ksi (3.1 t/cm^2). The yield stress of the reinforcing bar was 40.6 ksi (2.8 t/cm^2).

Concrete. The design strength of the concrete was 3.0 ksi (210 kg/cm^2). The compressive strength of the concrete ranged from 3.0 ksi (207 kg/cm^2) to 3.2 ksi (217 kg/cm^2) at test day.

Length. The length of the specimens was 19.7 in. (500 mm).

<u>SRC columns 88-92</u>

General. As part of a test series that comprised both columns and beam-columns, Han and Kim (1995) tested 5 SRC columns. The test parameters varied included the steel ratio (A_s/A_g), and a slenderness ratio (λ=15.2, 26, 36.8, 47.6). The stirrup diameter used was 0.2 in. (5 mm) at 2 spacing of 3.2 in. (80 mm) as well as at an interval of 1.6 in. (40 mm) at both ends to prevent local buckling due to concentration of the stress. The depth of the columns was 6.3 in. (160 mm). The ratio of structural steel to gross area of cross section ranged from 3.8 % to 8.6 %.

Steel. The average yield stress of structural steel was 45.9 ksi (3.2 t/cm^2) , and the modulus of elasticity ranged from 27507 ksi (1801 t/cm^2) to 27753 ksi (1914 t/cm^2). The yield stress of reinforcing bars was 40.6 ksi (2.8 t/cm^2), and the modulus of elasticity was 25375 ksi (1,750 t/cm^2).

Concrete. The design strength of the concrete was 3.0 ksi (210 kg/cm^2). The actual compressive strength of the concrete was between 3.0 ksi (207 kg/cm^2) and 3.2 ksi (217 kg/cm^2) .

Length. The lengths of the specimens were 27.6 in. (700 mm), 47.2 in. (1200 mm), 66.9 in.(1700 mm),and 82.2 in. (2200 mm). The support conditions reported are hinges at the ends of the member.

3.2.2 SRC Beam-Columns

<u>SRC Beam-columns 84-99</u>

General. The behavior of slender tied SRC beam-columns was studied with 16 specimens in which second order effects were significant (Mirza et al., 1997). The loading system consisted of concentric axial load and a pair of transverse loads acting simultaneously on pin-ended specimens. Since the specimens were cast and tested horizontally, they were also subjected to uniformly distributed transverse loads due to the self-weight of the specimen. All specimens had the same structural steel shape (HE100A) and overall dimensions of the concrete cross section (9.45 in. (240 mm) x 9.45 in. (240 mm)), which was square and symmetrical about both axes. Loading included combinations of axial and transverse forces producing a wide range of different external eccentricities. Observations from the physical tests indicated that for static loads the bonding condition at the interface of steel rib connectors and surrounding concrete has a small effect on ultimate strength. The tests also showed that the ACI 318 assumption of maximum usable strain of 0.003 at the concrete extreme compression fibers near ultimate load is valid for such beam-columns.

Steel. Structural steel with yield stresses of 42.5 ksi (293.4 MPa) and 55.1 ksi (311.4 MPa) were used. The ratio of cross-sectional area of structural steel to the area of overall concrete cross section was 4.2%. A nominal amount of longitudinal and transverse reinforcing steel was provided for all specimens. Near the specimen ends, the volumetric ratio of the ties was increased to approximately 2.65% and the tie spacing was decreased to 2 in. (50 mm). Nominal yielding strength of the reinforcing steel ranged from 81.9 ksi (565 MPa) to 91.9 ksi (634 MPa). The modulus of elasticity of steel section, ribs, and reinforcing bars was taken as 29000 ksi (200,000MPa).

Concrete. The concrete strength was measured using standard (5.9 in. x 5.9 in. x 5.9 in. (150 mm x 150 mm x 150 mm)) cubes. The strength of concrete was the average of three cube tests. Concrete strength was in the range of 4.5 ksi (31 MPa) to 5.1 ksi (34 MPa). For equivalent cylinder strength, a conversion factor of 0.81 was used for strength analyses.

Length. The effective length of all specimens was 157.6 in. (4000 mm).

<u>SRC Beam-columns 100-105</u>

General. Eight SRC beam-columns were tested by Roik and Diekman (1989) to study the effect of casting sequence on composite columns and check the rules for the design of composite columns given in DIN 18 806 Part 1. The application of this latter design method is limited to double-symmetric cross sections and does not include the case of a subsequent casting of loaded steel columns. In this study, loaded steel columns are transformed into composite columns by a subsequent casting of concrete. The results of some load capacity tests on subsequently and partially cast composite columns are given. They serve as a basis for a simple calculation model by which the existing concept in DIN 18 806 can be extended. All columns were bent about their strong axis. All columns, except No.13, were loaded up to 70% of their limit load. Column No.13 was loaded up to 100% of its limit load.

Steel. Structural steel sections with yield stress of 35.8ksi, 40.0 ksi, 41.3 ksi, and 58 ksi were used.

Concrete. The test specimens were cast in a flat position to achieve the best concrete placement possible. All composite columns were filled with concrete at the same time. All columns were cast of concrete with consistency K2 (normal concrete) and B35 characteristics. Concrete with compressive strengths of 5.2 ksi and 6.8 ksi was used.

Length. The effective length of specimens varied from 118.2 in. to 197 in..

SRC Beam-columns 106-109

General. As part of a test series that comprised both columns and beam-columns, Han et al. (1992) tested 4 SRC beam-columns. The test parameters varied included the cross section, ratio of structural steel to gross area of cross section and ratio of reinforcing hoops to gross area of cross-section. The depths of the columns were 6.3 in. (160 mm), representing a 1/3 scale specimen. The ratio of structural steel to gross area of the cross section ranged from 3.8 % to 8.6 %. The eccentricity-to-depth ratio of the specimens was 0.13.

Steel. Three tensile coupons were cut from the flange and web of each steel section, reinforcing bars, and hoops in accordance with Korean standard B 0801. The yield stress of the flange ranged from 42.5 ksi (2.9 t/cm^2) to 43.2 ksi (3.0 t/cm^2), and the yield stress of the web ranged from 33.6 ksi (2.3 t/cm^2) to 44.8 ksi (3.1 t/cm^2). The yield stress of the reinforcing bars was 40.6 ksi (2.8 t/cm^2).

Concrete. The design strength of the concrete was 3.0 ksi (210 kg/cm^2). The actual compressive strength of the concrete ranged from 3.0 ksi (207 kg/cm^2) to 3.2 ksi (217 kg/cm^2) at test day.

Length. The length of the specimens was 19.7 in. (500 mm).

<u>SRC Beam-columns 110-124</u>

General. As part of a test series that comprised both columns and beam-columns, Han and Kim (1995) tested 15 SRC beam-columns. The test parameters varied included the steel ratio (A_s/A_g), and slenderness ratio (λ=15.2, 26, 36.8, 47.6). The stirrup diameter used was 0.2 in. (5 mm) at 2 spacing of 3.2 in. (80 mm) as well as at 1.6 in. (40 mm) at both ends to prevent damage from local buckling. The depth of the columns was 6.3 in. (160 mm). The ratio of structural steel to gross area of cross section ranged from 3.8 % to 8.6 %. The eccentricity-to-depth ratio of the specimens was between 0.13 and 0.5.

Steel. The average yield stress of structural steel was 45.9 ksi (3.2 t/cm^2), and the modulus of elasticity ranged from 27507 ksi (1,801 t/cm^2) to 27753 ksi (1,914 t/cm^2). The yield stress of reinforcing bar was 40.6 ksi (2.8 t/cm^2), and the modulus of elasticity was 25375 ksi (1,750 t/cm^2).

Concrete. The design strength of the concrete was 3.0 ksi (210 kg/cm^2). The compressive strength of the concrete was between 3 ksi (207 kg/cm^2) and 3.2 ksi (217 kg/cm^2).

Length. The lengths of the specimens were 27.6 in. (700 mm), 47.2 in. (1200 mm), 66.9 in. (1700 mm),and 82.2 in. (2200 mm). The support conditions reported are hinges at the ends of the member.

3.2.3 CCFT Columns

<u>CCFT Columns 205-210</u>

General. Six composite columns with yield stress of 80 ksi were tested as part of this study (Kenny et al., 1994). Five of the members sustained ultimate load in excess of the predicted nominal strength, with one exhibiting a test load slightly less than nominal. Steel tubes maintained their cross-sectional shape, and no distress of the concrete was evident. Therefore, authors recommended that the 55 ksi yield stress limit given by the`1979 Structural Stability Research Council (SSRC) paper be increased to 80 ksi. Concrete that is confined by the continuous shell provided by steel tubes was shown to be capable of sustaining substantially higher strains than 0.0018 before either material fails. The scope of the work did not allow for the development of a new composite columns strength model. However, the conservatism of the current criteria point to a significant need for further studies of this problem. In particular, a strength model that incorporates concrete confinement effects will be most useful.

Steel. The primary objective of this work was to extend the existing design equations to include higher strength steel materials, specifically, steel with yield stress higher than 55 ksi. Therefore, steel tubes with nominal yielding stresses of 98.9 ksi and 86.1 ksi were used.

Concrete. Two cylinders of the material used in filling the pipes were tested to determine the 28 day unconfined compressive strength. The compressive strength of the concrete got from the results of material test was 5.7ksi and 5.5 ksi.

Length. For every diameter there were three sections of 3-ft long and one section of 10-ft long.

CCFT Columns 211-215

General. Five tubes (steel and concrete loaded simultaneously) were tested by O'Shea and Bridge (1997). Five diameter-to-thickness ratios ranging from 55 to 200 were selected for the tests to cover tube behavior ranging from fully effective tubes to tubes where local buckling dominates the behavior. A high strength concrete (16.5 ksi) was used as the infill. The potential strength enhancement and the possible improved ductility from confinement of high strength concrete used in axially loaded tubes were examined.

Steel. To cover the range of wall thickness used in the specimens, three different types of steel were used. For the commercial specimens with D/t = 55, the steel was cold formed during tube manufacture. For D/t = 95 and 120, a hot rolled steel sheet plate was used. For D/t = 165 and 200, a cold rolled steel sheet was used. It was found that the proof stress for the tube was higher than for the unformed plates as a result of the manufacturing process. The residual stresses in all manufactured steel tube sizes have not been measured as the authors have found that the membrane residual stresses were small for tubes produced using a similar manufacturing process.

Concrete. Only commercially available materials with normal mixing and curing techniques were used. A superplasticizer (Rheobulid 1000) was added during the mixing process to provide the desired workability and slump of 3.9 in. (100 mm). The average concrete compressive strength was 16.5 ksi (113.5 MPa).

Length. A length-to-diameter ratio of 3.5 was chosen to reduce end effects and yet still ensure that the specimens would be stub columns with little effects from column slenderness.

<u>CCFT Columns 216-226</u>

General. The behavior of very slender concrete filled steel tubular columns and hollow tubular columns subjected to axial loading was experimentally investigated (Han and Yan, 2000). A total fifteen specimens including eleven CCFT and four hollow tubular columns were studied. Experimental results indicate that the concrete delays the development of local buckling of the steel tube and thus increases its compressive load capacity. In general, concrete infill can not only enhance the ultimate strength of the columns, but can also delay the local buckling of the steel. The loading capacity of the CCFT column can be conservatively predicted by using of the recommendations of the "Chinese Design Code for steel Concrete Composite Structures DLT/T 5085-1999" for normal length. The ends of the steel tubes were cut and machined to the required length. The insides of the tubes were wire brushed to remove any rust and loose debris present. The specimens all failed in the center by rupture of the steel casing in the tension zone after substantial cracking of the concrete in the tension zone and buckling of the steel in the compression zone.

Steel. The steel had a yield strength of 59.5 ksi (348.1 MPa).

Concrete. Two types of concrete were used. The concrete mixes were designed for compression cube strengths at 28 days of 3.8 ksi (31.8 MPa) and 5.7 ksi (46.8 MPa) respectively. For each batch of concrete mixed, three 3.9 in. (100 mm) cubes were also cast and cured in conditions similar to the related specimens.

Length. Columns with slenderness ratios greater than 120 have been studied. The length of the columns ranged from 138.3 in. (3510 mm) to 163.8 in. (4158 mm) with a constant diameter of 4.3 in. (108 mm).

<u>CCFT Column 227</u>

General. As part of a test series that comprised both columns and beam-columns, Kilpatric and Rangan (1997) tested one CCFT column. The specimen was constructed using a commercially available circular hollow steel tube which was manufactured by cold forming and high frequency electric resistance welding. Both ends of the column were clamped specially made hardened knife-edge assemblages which had been previously located on the top and bottom plates of the testing machine. Both a prediction of column strength and a force deflection response were obtained using a deformation control method of analysis and conclusions were drawn on the basis of comparisons between measured and predicted data.

Steel. Tests showed that the 4 in. (101.5 mm) x 0.09 in. (2.4 mm) tube had a 0.2% offset tensile strength of 59.5 ksi (410 MPa) and an ultimate tensile strength of 68.9 ksi (475 MPa).

Concrete. The 0.4 in. (10 mm) aggregate high strength concrete was commercially supplied. Standard tests conducted on ten 3.9 in. (100 mm) diameter cylinders were tested to determine the compressive strength of the concrete which was found to be an average of 13.9 ksi (96 MPa).

Length. The length of each column was kept constant at 85.7 in. (2175 mm) center to center of male knife edges.

CCFT Columns 228-233

General. As part of a test series that comprised both columns and beam-columns, Matsui et al. (1997) tested six CCFT columns. The diameter-to-thickness ratios of the steel plate element were 33 and 37, respectively. The design method according to the AIJ standards for concrete filled steel tubular structures is summarized. A modified AIJ method has been proposed by the authors. The difference between the AIJ and the modified AIJ method is only in the strength of the concrete column. In the modified AIJ method, a high accuracy concrete column strength based on a numerical analysis is proposed.

Steel. The material of the steel portion was mild steel STK400 (circular), Japanese Industrial Standards). Tests showed that the circular tube had a yield strength of 51.2ksi (3.6 t/cm^2).

Concrete. The average compressive strength of concrete for the circular columns was 4.6 ksi (417 kg/cm^2).

Length. The effective length of columns ranged from 26 in. (600 mm) to 195.3 in.

(4956 mm).

CCFT Columns 234-246

General. Giakoumelis and Lam (2003) tested 13 short circular columns under axial load. They compared high strength concrete and normal strength CFT columns, and greased and non-greased CFT columns. The strength of the columns were calculated and compared to the Eurocode 4, Australian standards and American codes. Eurocode 4 gave a good result for both high and normal strength concrete column. The diameter of the tubes was 4.5 in. (+/−0.01), and the depth-to-thickness ratio (D/t) ranged from 22.9 to 30.5.

Steel. The yield stresses of the tubes ranged from 49.7 ksi (343 MPa) to 52.9 ksi (365 MPa).

Concrete. Concrete with normal mixing and curing techniques and produced from commercially available materials was used. Standard cube test indicated 4.4 ksi (30 N/mm^2), 8.7 ksi (60 N/mm^2) and 14.5 ksi (100 N/mm^2) compressive strengths.

Length. These series of tests were stub column tests. Thus, the effective length of the columns was very short. The length of the columns was 11.8 in..

CCFT Columns 247-249

General. As part of a test series that comprised both circular CFT columns and rectangular CFT columns, Schneider (1998) constructed and tested circular CFT columns under concentric axial compression. The tests were divided into three groups: CCFT columns, square tube columns and rectangular tube columns. Three were circular steel tubes. The test parameters varied included the shape of the steel tube and the D/t ratio. The confinement of the concrete core and the influence of the compression of the steel tube were studied. The diameters of the tubes were 5.5 in. (140 mm), and the depth-to-thickness ratio (D/t) ranged from 21 to 46.9. The ratio of the steel and total area ranged from 8.3% to 18.2%.

Steel. All steel tubes were cold-formed carbon steel with the specified yield strength of 45.97 ksi (317 MPa). All tubes were welded and annealed to relieve residual stresses. Each tube was supplied longer than needed for the CFT test to provide enough material for a tensile coupon test.

Concrete. To get the 28-day target strength of 2.9 ksi (20 MPa) and 3.0 in. (75 mm) slump, type I Portland cement, sand and a maximum aggregate size of 0.4 in. (9.5 mm) were mixed. The concrete was mixed in four batches. Each batch provided four standard concrete cylinders. Each cylinder was tested within 2 days to get strength of the concrete and elastic modulus. The actual cylinder strengths ranged from 3.5 ksi (23.8 MPa) to 4.1 ksi (28.2 MPa).

Length. The effective length of these columns was 24 in. (610 mm). A stiffened end cap was attached at the base of each steel tube.

<u>CCFT Columns 250-252</u>

General. As part of a test series that comprised both columns and beam-columns, Han and Yao (2003) tested three CCFT columns. The diameter of the tubes was 4.7 in. (120 mm). The depth-to-thickness ratio (D/t) was 45.3. The tubes were manufactured from mild steel sheet, with four plates cut from the sheet, tack welded into a square shape and then welded with a single bevel butt weld at the corners. The influence of the preload ratio, slenderness ratio, steel ratio and the strength of the materials were investigated.

Steel. Three tension coupons were cut from a steel sheet to determine the steel material properties. From these tests, the yield strength of the steel was found to be 49.3 ksi (340 N/mm^2) and the modulus of elasticity was 30,015 ksi (207,000 N/mm^2).

Concrete. To determine the strength of the concrete cubes, they were tested at 28 days. The strength of the concrete ranged from 2.9 ksi (20.1 MPa) to 5.2 ksi (36 MPa). The mix proportions of the concrete were 31.2 pcf (500 kg/m^3) of cement, 12.2 pcf (195 kg/m^3) of water, 30.0 pcf (480 kg/m^3) of sand and 76.5 pcf (1225 kg/m^3) of coarse aggregate.

Length. This series of tests ranged in length from 14.2 in. (360 mm) to 55.1 in. (1400 mm). The ends of the steel tubes were cut and machined to the required length. Each tube was then welded to a square steel base plate of 0.6 in. (16 mm) nominal thickness.

CCFT columns 253-261

General. Columns 253-261 were tested by Roeder and Cameron (1999). The diameters of the tubes ranged from 10.3 in. (261 mm) to 23.8 in. (604 mm). The depth-to-thickness ratio (D/t) varied from 19.4 to 108. This paper shows that shrinkage can be very detrimental to bond stress capacity, and the importance of shrinkage depends on the characteristics of the concrete, the diameter of the tubes and the D/t ratio.

Steel. The yield stress of the steel was not reported in this paper. A typical yield stress (40 ksi) was used for analysis.

Concrete. The compressive strength of the concrete ranged from 6.4 ksi (43.9 MPa) to 6.8 ksi (47.3 MPa). Age of concrete at testing ranged from 24 to 57 days to determine effect of the shrinkage.

Length. This series of tests ranged in length from 31.9 in. (810 mm) to 75.9 in. (1927 mm).

CCFT Columns 262-294

General. As part of a test series that comprised both columns and beam-columns, O'Shea and Bridge (2000) tested 33 CCFT columns. The diameters of the tubes ranged from 6.5 in. (165 mm) to 7.5 in. (190 mm). The diameter-to-thickness ratio (D/t) was between 58.5 and 220.9. In this test series, tested loading conditions include axial loading of the steel only, axial loading of the concrete only, and simultaneously loading of the concrete and steel. Test specimens were divided into four types: concrete loaded columns, axially loaded thin walled circular CFT columns, columns with no local buckling, and eccentrically loaded thin walled circular CFT columns.

Steel. The yield stress of the tubes ranged from 26.9 ksi (185.7 MPa) to 52.7 ksi (363.3 MPa). The maximum yield stress was almost twice the specified minimum yield stress.

Concrete. The internal concrete had nominal unconfined cylinder strengths of 7.3 ksi (50 MPa), 11.6 ksi (80 MPa), and 17.4 ksi (120 MPa).

Length. The length of the columns ranged from 22.1 in. (562.5 mm) to 26.1 in. (664 mm). The tests specimens were short with a length-to-diameter ratio of 3.5. Thick end plates were used during the tests to ensure that the load applied at a constant eccentricity resulted in a linear strain.

<u>CCFT Columns 295-302</u>

General. As part of a test series that comprised of both circular CFT columns and rectangular columns, Kang et al. (2002) tested 8 circular CFT columns subjected to concentric load. The test parameters varied included the steel shape (circular and square), the depth-thickness-ratio and in-filled concrete. The diameter-to-thickness ratios (D/t) were 27.3, 23.4, 31.3 and 43.5. The depths of the columns were 3.0 in. (76.3 mm), 3.5 in. (89.1 mm), 4.0 in. (101.6 mm) and 4.5 in. (114.3 mm).

Steel. Eight types of steel plates were used (SPS400). Three coupons from each plate were taken and were tested by the Korean standard B 0801. The yield stress of the steel tube was reported as 40.2 ksi (2.77 t/ cm^2), 40.6 ksi (2.80 t/ cm^2), 49.7 ksi (3.4 t/ cm^2), and 52.9 ksi (3.7 t/ cm^2).

Concrete. The target strength of the concrete was 7.3 ksi (500 kg/cm^2). Three cylinders were made from each batch, and tested in accordance with Korean standard F 2403 at 28 days. There were two types of concrete: normal Portland cement concrete and polymer cement concrete. The mixed proportions for normal concrete were 29.96 pcf (480 kg/m^3) of cement, 10.2 pcf (163 kg/m^3) of water, 52.9 pcf (848 kg/m^3) of fine aggregate and 50.9 pcf (815 kg/m^3) of coarse aggregate . The mixed proportions for polymer concrete were 5.6 pcf (90 kg/m^3) of polymer, 28.1 pcf (450 kg/m^3) of cement, 7.9 pcf (126 kg/m^3) of water, and 45.1 pcf (723 kg/m^3) of fine aggregate and 52.9 pcf (848 kg/m^3) of coarse aggregate . The concrete compressive strength was 7.9 ksi (542 kg/cm^2) for the normal concrete and 6.8 ksi (467 kg/cm^2) for the polymer modified concrete.

Length. The length of the columns ranged from 9.0 in. (22.9 mm) to 13.5 in. (34.3 mm). The length of the columns was three times the section depth.

CCFT Columns 303-312

General. A total of 10 columns were tested to investigate the bond stress and axial load distribution between a steel tube and in filled concrete by Woo and Kim (2002). Loads were transferred by bond stress between in filled concrete and the interior surface of a steel tube, and these tests were conducted to elucidate the load transfer mechanism. The diameter of steel tubes was 12.5 in. (318.5 mm). The diameter-to-thickness ratio was 46.2.

Steel. The average yield stress of the steel tube was 55.8 ksi (3.9 t/ cm^2) and the ultimate stress was 78.9 ksi (5.4 t/ cm^2).

Concrete. Two types of concrete, normal concrete and high workable concrete, were used. The concrete compressive strength at 28 day for the normal concrete was 3.9 ksi (268 kg/cm^2) and that of the high workable concrete was 3.5 ksi (243 kg/cm^2).

Length. The length of the columns ranged from 14.1 in. (358.5 mm) to 39.2 in. (995.5 mm).

3.2.4 CCFT Beam-Columns

CCFT Beam-Column 129-133

General. For this test series, the test specimens were circular columns of 9.8 in. (250 mm) diameter and had eccentricities ranging from 0.9 in. (23 mm) to 1.2 in. (33 mm) (A.Kvederas and A. Tomaszewicz, 1994). This paper presents an investigation of the effect of a thin steel wall on the strength and deformation behavior of circular concrete columns. In this construction concept the wall thickness of the steel tubes did not meet the thickness of typical composite columns. The interaction between concrete core and steel hollow section in compression and the influence on ductility and strength are discussed with reference to the test results.

Steel. A thin 0.08 in. (2 mm) steel plate bent into the shape and longitudinally welded in tubular form was used. The yield stress was 34.8 ksi (280 MPa). Columns S3 and S5 were without longitudinal reinforcement, while the other columns in the series were longitudinally reinforced with six K500T reinforcing steel bars with diameter 0.6 in. (16 mm).

Concrete. Columns S1-S3 were cast using concrete grade C85 concrete, while S4 and S5 were cast using grade C45 concrete. Concrete compressive strength ranged from 5.2 ksi (35.7 MPa) to 10.5 ksi (72.6 MPa).

Length. Columns S1-S5 had a length of 86.7 in. (2200 mm).

CCFT Beam-column 134

General. The test series consisted of 11 slender columns tested to failure under eccentric axial loading (Johansson et al., 2000). Nine of these columns were circular hollow steel sections filled with concrete and two were unfilled steel tubes. Among them, three columns were appropriate for the database. Finally, only one column was used for the database, because there was one test result at each test series. The load was applied to the concrete section, to the steel section, or to the entire section. The columns were hinged at both ends and loaded with a compressive axial force applied with an initial eccentricity of 0.4 in. (10 mm). The end eccentricity was equal at both ends and the column was loaded uniaxially. An experimental study of the structural behavior of circular composite columns and results from nonlinear finite element analysis are presented. Eleven full-scale columns were tested to failure under eccentric axial loading. The parameters varied in this investigation were the means of load application to the top of the columns and the bond between the concrete core and the steel section. The results of finite element analysis correlated well with the results of the tests. There was almost no difference in the loading bearing capacity when the load was applied to the concrete section compared with when the load was applied to the entire section. However, it can be observed that the load bearing capacity was drastically reduced when the load was applied to the steel section only. There was no significant change in the structural behavior when the bond condition was changed.

Steel. The yield stress of the steel tubes was reported as 62.8 ksi (433 MPa) which is the average of five tensile tests.

Concrete. The compressive cylinder strength was 9.4 ksi (64.5 MPa).

Length. The length of the columns was 98.4 in. (2500 mm) with circular 6.3 in. (159 mm) diameter cross section.

CCFT Beam-column 135-136

General. As part of a test series that comprised both columns and beam-columns, Kilpatric and Rangan (1997) tested two CCFT beam-columns. All specimens were constructed using commercially available circular hollow steel tube which was manufactured by cold forming and high frequency electric resistance welding. Both ends of each column were clamped to specially made hardened knife-edge assemblages which had been previously located on the top and bottom plates of the testing machine. The eccentricity of the applied compressive force was obtained by displacing the end of the column laterally from the axis of the testing machine. The eccentricity was varied from that producing single curvature bending to double curvature bending while the length of the column was kept constant. Prediction of column strength and force deflection response were obtained using a deformation control method of analysis and conclusions were drawn on the basis of comparisons between measured and predicted data.

Steel. Tests showed that the 4 in. x 0.94 in. (101.5 mm x 2.4 mm) tube had a 0.2% offset tensile strength of 59.45 ksi (410MPa) and an ultimate tensile strength of 68.89 ksi (475 MPa).

Concrete. The 0.4 in. (10 mm) aggregate high strength concrete was commercially supplied. Standard tests conducted on ten 3.9 in. (100 mm) diameter cylinders yielded an average of 13.9 ksi (96 MPa).

Length. The length of each column was kept constant at 85.7 in. (2175 mm) center to center of male knife edges.

CCFT Beam-column 137-154

General. As part of a test series that comprised both columns and beam-columns, Matsui et al. (1997) tested 18 CCFT beam-columns. The diameter-to-thickness ratios of the steel plate element were 33 and 37, respectively. The design method according to the AIJ standards for concrete filled steel tubular structures is summarized. A modified AIJ method has been proposed by the authors. The difference between the AIJ and the modified AIJ method is only in the strength of the concrete column. In modified AIJ method, high accuracy concrete column strength based on a numerical analysis is proposed.

Steel. The material of the steel portion was mild steel (STK400(circular), Japanese Industrial Standards). Tests showed that the circular tube had a yield strength of 51.2ksi (3.6 t/cm^2).

Concrete. The average compressive strength of concrete for the circular columns was 4.62 ksi (417 kg/cm^2).

Length. The The effective length of columns ranged from 26 in. (600 mm) to 195.3 in. (4956 mm).

CCFT Beam-columns 155-160

General. Total 6 CFT beam-columns were tested under cyclic lateral loading and constant axial loading by Elremaily and Azizinamini (2002). The test parameters included the level of axial load, the diameter-to-thickness ratio of the steel tube, and the concrete compressive strength. The diameters of steel tube used were 12.8 in. (324 mm). The diameter-to-thickness ratio ranged from 34.1 to 50.6.

Steel. The material properties of the steel tubes were measured by conducting coupon tests and the average yielding stress of steel tubes was found to be 54 ksi (372 MPa).

Concrete. The target concrete compressive strength ranged from 5 ksi (34 MPa) to 15 ksi (103 MPa). The concrete strength for each specimen was determined at the test day by testing concrete cylinders sampled from the concrete batch used to fill the test specimen.

Length. The supported span of the columns was 86 in. (2184 mm). The size of the specimen was chosen to be approximately two-thirds scale compared with the column size required for a typical building. The specimen was capped on both ends with rigid steel caps to uniformly distribute the applied axial load.

<u>CCFT Beam-columns 161-164</u>

General. As part of a test series that comprised both columns and beam-columns, Han and Yao (2003) tested 4 CCFT beam-columns. The diameters of the tubes were 4.7 in. (120 mm). The depth-to-thickness ratios (D/t) were 45.3. The tubes were manufactured from mild steel sheet, with four plates cut from the sheet, tack welded into a square shape and then welded with a single bevel butt weld at the corners. The load-deflection behavior of concrete filled columns with the steel tubes subjected to preload is described. The influence of the preload ratio, slenderness ratio, steel ratio and the strength of the materials were compared.

Steel. Three tension coupons were cut from steel sheet to determine the steel material properties. From these tests, the yielding strength of the steel was found to be 49.3 ksi (340 N/mm^2) and the modulus of elasticity was 30,015ksi (207,000 N/mm^2).

Concrete. To determine the strength of the concrete cube, it was tested at 28 days.

The strength of the concrete ranged from 2.9 ksi (20.1 MPa) to 5.2 ksi (36 MPa). The mix proportions of the concrete were 31.2 pcf (500 kg/m^3) of cement, 12.2 pcf (195 kg/m^3) of water, 30.0 pcf (480 kg/m^3) of sand, and 76.5 pcf (1225 kg/m^3) of coarse aggregate.

Length. This series of tests ranged from 14.2 in. (360 mm) to 55.1 in. (1400 mm). The ends of the steel tubes were cut and machined to the required length. Each tube was then welded to a square steel base plate of 0.6 in. (16 mm) nominal thickness.

CCFT Beam-columns 165-187

General. As part of a test series that comprised both columns and beam-columns, O'Shea and Bridge (2000) tested 23 CCFT beam-columns. The diameters of the tubes ranged from 6.5 in. (165 mm) to 7.5 in. (190 mm). The diameter-to-thickness ratio (D/t) were between 58.5 to 220.9. In this test series, tested loading conditions include axial loading of the steel only, axial loading of the concrete only, and simultaneously loading of the concrete and steel both axially and at small eccentricities. Test specimens were divided into three types which were concrete loaded columns, axially loaded thin walled circular CFT columns, columns with no local buckling, and eccentrically loaded thin walled circular CFT columns.

Steel. The yield stress of the tubes ranged from 26.9 (185.7 MPa) to 52.7 ksi (363.3 MPa). Maximum yielding stress is almost twice of minimum yielding stress.

Concrete. The internal concrete had nominal unconfined cylinder strengths of 7.3 ksi (50 MPa), 11.6 ksi (80 MPa), and 17.4 ksi (120 MPa).

Length. The length of the columns ranged from 22.1 in. (562.5 mm) to 26.1 in. (664 mm). The tests specimens were short with the length-to-diameter ratio of 3.5. Thick end plates were used during the tests to ensure that the load applied at a constant eccentricity resulted in a linear strain.

<u>CCFT Beam-columns 188-198</u>

General. Beam-columns 188-198 were tested by Jung et al. (1994). The diameters of steel tube were 10.5 in. (267.4 mm). The depth-to-thickness ratio (D/t) were 50.9 and 66.9. To evaluate the elasto-plastic behavior of concrete filled circular tube column under axial force and bending moment, reverse moment at both ends of the specimens were monotonically and cyclically applied under constant axial force, to verify the effectiveness of the studs in maintaining composite section, the studs are attached to the inside surface of the steel tube.

Steel. Three tension coupons from each steel plate were cut from the steel sheets to determine the steel material properties. Thus, total six coupons were tested according to KS B 0801. The yielding stress of the steel tube was 35.4 ksi (2.4t/cm^2).

Concrete. The design compressive strength of the concrete was 4.4 ksi (300 kg/cm^2)

and 5.8 ksi (400 kg/cm^2). Ten concrete cubes were manufactured from each batch. A total of twenty cubes were tested. The compressive strength of concrete was ranged from 4.79 ksi (330 kg/cm^2) to 6.38 ksi (440 kg/cm^2).

Length. The length of the columns was 51.2 in. (1300 mm). Both ends of the specimens were supported by an end plate.

3.2.5 RCFT Columns

<u>RCFT Columns 101-103</u>

General. In this test series, (Song and Kwon, 1997) to assure uniform compression and prevent the eccentricity, very thick loading plates were attached at each end (top and bottom) of test specimens. Preliminary tests were carried out within the elastic range by adjusting the loading plate, based on the measurements of strain and displacement.

The failure of the concrete filled columns showed an asymmetric buckling mode against the axes of the cross section. All steel panels buckled outward because the buckling towards the inside was prevented by the filled in concrete. After the local buckling of the plates, deformation rapidly increased and cracks occurred in the web.

An experimental study on the behavior of concrete filled steel box stub columns was performed. In addition, simple formulas for design of composite column were proposed based on the test results.

Steel. The yield strength of the steel was 45.5 ksi (3.2 t/cm^2). It was a little higher than the nominal strength because of the welding and cutting.

Concrete. To determine the compressive strength of the concrete, 15 cylinders (3.9 in. (100 mm) diameter x 11.4 in. (290 mm) high) were cast from the same concrete used inside the concrete filled column. The average value obtained for 15 cylinders was 4.37 ksi (307 kg/cm^2).

Length. The lengths of columns were varied from 15.4 in. (390 mm) to 26 in. (660 mm).

RCFT Columns 104-109

General. As part of a test series that comprised both columns and beam-columns, Matsui et al. (1997) tested six rectangular CFT columns. The depth-to-thickness ratio of the steel plate element was 35.1. In this paper, the design method according to the AIJ standards for concrete filled steel tubular structures is summarized. A modified AIJ method has been proposed by the authors. The difference between the AIJ and the modified AIJ method is only in the strength of the concrete column. In modified AIJ method, high accurate concrete column strength based on a numerical analysis is proposed.

Steel. The material of the steel portion was mild steel (STKP400(square), Japanese

Industrial Standards). Tests showed that the square tube had the yield strength of 59.7 ksi ($4.2t/cm^2$).

Concrete. The average compressive strength of concrete for square columns was 5.93 ksi ($325 kg/cm^2$)

Length. The The effective length of columns ranged from 23.62 in. (600 mm) to 177.3 in. (4956 mm).

<u>RCFT Columns 110-120</u>

General. As part of a test series that comprised both circular CFT columns and rectangular CFT columns, Schneider (1998) constructed and tested rectangular CFT columns under concentric axial compression. Tests are divided three groups which are CCFT columns, square tube columns and rectangular tube columns. This paper presented and experimental study on short, concentrically loaded, concrete filled steel tube columns. Variables for this experiment were the shape of the steel tube and the D/t ratio. The section size of the tubes were 5 in. (127 mm) x 5 in. (127 mm), 3 in. (76 mm) x 6 in. (152 mm), and 4 in. (102 mm) x 6 in. (152 mm) and the depth-to-thickness ratio (D/t) ranged from 17 to 40.4. The ratio of the steel and total area ranged from 9.5 % to 21.7 %.

Steel. All steel tubes were cold-formed carbon steel with the yield strength which ranged from 46.7 ksi (322 MPa) to 62.4 ksi (430 MPa). All tubes were seam welded and annealed to relieve residual stresses.

Concrete. To get the 28-day target strength of 2.9 ksi (20 MPa) and a 3 in. (75 mm) slump, type I Portland cement, sand and a maximum aggregate size of 0.4 in. (9.5 mm) were mixed. The concrete was mixed in four batches. Each batch provided four standard concrete cylinders. Each cylinder was tested within 2 days to get strength of the concrete and elastic modulus. The actual cylinder strengths ranged from 3.5 ksi (23.8 MPa) to 4.1 ksi (28.2 MPa).

Length. The effective length of the columns was 24 in. (610 mm). A stiffened end cap was attached at the base of the specimens.

RCFT Columns 121-143

General. As part of a test series that comprised both rectangular CFT columns and beam-columns, Han and Yao (2002) tested 23 rectangular CFT columns. The depths of the columns were 5.1 in. (130 mm), 7.7 in. (195 mm), 9.5 in. (240 mm), and 14.2 in. (360 mm). The depth-to-thickness ratios (D/t) were 34, 45.3, 49.1, and 90.6. The ratio of the steel and total area ranged from 3.6% to 9.6%

Steel. The steel tubes were all manufactured from mild steel sheet, with four plates were cut from the sheet, tack welded into a rectangular shape and then welded with a single bevel butt weld at the corners. Strips of the steel tubes were tested in tension in accordance with Chinese standard GB2975. Three coupons were taken from each face of the steel tube. From these tests, the average yield strength of the tube was found to be 49.3 ksi (340.1 MPa) and the modulus of elasticity was about 30015 ksi (207000 MPa). The end of steel tubes were cut and machined to the required length. The insides of the tubes were wire brushed to remove rust and loose debris present. The deposits of grease and oil, if any, were cleaned away. Each tube was welded to a rectangular steel base plate 10 mm thick. The specimen was placed upright to air-dry until testing occurred. The present study is an attempt to study the influence of concrete compaction on the strength of concrete filled steel RHS columns. Thirty five concrete filled steel RHS columns were tested to investigate the influence of concrete compaction methods compared to member capacities of the composite columns. The main parameters which varied in the tests were the column section depth-to-width ratio, tube depth to thickness ratio, load eccentricity and column slenderness.

Concrete. The concrete mix was designed for compressive cube strength at 28 days with the Chinese standard GBJ81-85, the average value being 3.7 ksi (253 MPa) The mixed proportions were 25.2 pcf (403 kg/m^3) of cement, 9.6 pcf (153 kg/m^3) of water, 35 pcf (561 kg/m^3) of sand and 80.1 pcf (1283 kg/m^3) of coarse aggregate. The average cube strength at the time of the test was 3.4 ksi (23.1 MPa). In all thee concrete mixes, the fine aggregate used was silica-base, the coarse aggregate was carbonate stone. The specimen tests allowed for different conditions likely to arise in the manufacture of concrete: cured, well compacted with a poker vibrator and well compacted by hand. During curing, a very small amount of longitudinal shrinkage of 0.02 in. (4 mm)-0.03 in. (7 mm) or so occurred at the top of the column.

Length. The lengths of columns were 21.3 in. (540 mm), 30.7 in. (780 mm), 56.7 in. (1440 mm), and 92.1 in. (2340 mm). Both the end plate and the top plate were made of very hard and very high strength steel.

RCFT Columns 144-152

General. As part of a test series that comprised both rectangular CFT columns and beam-columns, Uy (2000) tested 9 rectangular CFT columns. The depth-to-thickness ratios (D/t) were 22, 32, and 42. The depths of the columns were 4.3 in. (110 mm), 6.3 in. (160 mm), and 8.3 in. (210 mm). This study included an extensive set of experiments on high strength steel box columns filled with concrete and a numerical model which is presented and calibrated. The main parameters were tube width, depth to thickness ratio, concrete compressive strength, yielding stress of steel tube, eccentricity, and local buckling.

Steel. Residual stress measurements were conducted using a combination of both electric strain gauges and mechanical strain gauges across the width of the plates. The yield stress of the steel tube was 108.8 ksi (750 MPa).

Concrete. To determine the average compressive strength of the concrete a series of standard cylinders were crushed throughout the testing period. The average of compressive strengths was given for different ages of testing. The reported compressive strengths of the concrete were 4.1 ksi (28 MPa), 4.4 ksi (30MPa), and 5.8 ksi (40 MPa),

Length. To ascertain a uniform loading surface, columns were cast in place with plates with plaster at either end. The eccentrically loaded columns were loaded using a knife-edge at both the top and bottom of the column. The length of the columns was 118.1 in. (3000 mm).

RCFT Columns 153-156

General. Columns 153-156 were tested by Uy (2002). The depth-to-thickness ratio (D/t) varied from 21.7 to 25. The depths of the columns were 2.6 in. (65 mm) and 3.0 in. (75 mm). Test parameters were the effect of the column slenderness (L/D), the plate slenderness (b/t), the steel yield strength, and the concrete compressive strength.

Steel. Traditionally mild structural steel with a yield stress of 36.3 ksi (250 MPa) and 50.8 ksi (350 MPa) was used. The yield stress of the steel tube was 58.0 ksi (400 MPa) and 65.3 ksi (450 MPa).

Concrete. Concrete compressive strengths in column construction generally start at about 3.6 ksi (25 MPa). Most modern reinforced concrete codes allow for strengths up to 7.3 ksi (50 MPa). Extensive research over the last decade has seen concrete compressive strength tested and applied up to 14.5 ksi (100 MPa). The concrete compressive strength of the specimens were 7.54 ksi (52 MPa) and 11.46 ksi (79MPa).

RCFT Columns 157-162

General. As part of a test series that comprised both rectangular CFT columns and beam-columns, Seo and Chung (2002) tested 5 rectangular CFT columns. The depth-to-thickness ratio (D/t) was 39.1. The depth of the columns was 4.9 in. (125 mm). Test parameters were the bucking length-section ratio and the eccentricity of the applied compressive load.

Steel. To determine the yielding stress and material properties of steel tube, tensile coupon tests and stub column tests were conducted. The yield stress of steel tube was 65.5 ksi (4.5 t/cm^2).

Concrete. The water-cement ratio was 26%. The mix proportions were 32.5 pcf (520 kg/m^3) of cement, 10.6 pcf (714 kg/m^3) of water, coarse aggregate 54.2 pcf (869 kg/m^3), and fine aggregate 44.6 pcf (714 kg/m^3). Design concrete compressive strength was 15.3 ksi (1057 kg/cm^2). The mean concrete compression strength at 28 day was 13.0 ksi (898 kg/cm^2). The mean concrete compression strength at test day was 13.9 ksi (960 kg/cm^2). To get high workability, silica fume was added.

Length. The length of the columns were 11.8 in. (300 mm), 19.7 in. (500 mm), 39.4 in. (1000 mm), 59.1 in. (1500 mm), 88.6 in. (2250 mm), and 147.6 in. (3750 mm). Support condition of the columns was hinge.

<u>RCFT Columns 163-171</u>

General. Columns 163-171 were tested by Kang et al. (2001). The depth-to-thickness ratio (D/t) was between 16.7 and 93.8. The depths of the columns were 7.9 in. (200 mm), 9.8 in. (250 mm) and 11.8 in. (300 mm). Test parameters were the depth-to-thickness ratios of steel tube and the ratio of concrete cylinder strength-to-yield stress of steel tube.

Steel. Five coupons were cut from the steel tube which had a 0.2 in. (5 mm) thickness. A coupon was cut from the steel tube which had 0.35 in. (9 mm) and 0.47 in. (12 mm) thickness. Strips of the steel tubes were tested in tension in accordance with Korean standard B 0801. The average yield strength of the tube was found to be 46.1 ksi (3.2 t/cm^2) and 52.8 ksi (3.6 t/ cm^2), and the modulus of elasticity ranged from 26956 ksi (1859 t/cm^2) to 37483 ksi (2585 t/cm^2).

Concrete. To determine the average compressive strength of the concrete, a total of 20 standard cylinders were crushed throughout the testing period in accordance with

Korean standard KSF 2404. The target strengths of the concrete were 3.1 ksi (210 kg/cm^2) and 4.4 ksi (300 kg/cm^2). The average of compressive strengths were 3.6 ksi (248 kg/cm^2) and 4.4 ksi (304 kg/cm^2).

Length. The lengths of the columns were 23.6 in. (600 mm) , 29.5 in. (750 mm), and 35.4 in. (900 mm). The length of the columns was three times that of the section depth.

RCFT Columns 172-177

General. Columns 172-177 were tested by Yang and Seo (1998). The depth-to-thickness ratio (D/t) was 25.1, 33.9 and 45. The depth of the columns was 3.9 in. (100 mm).

Steel. The steel tubes which were used in the test were all manufactured products (SPSR400). Three coupons were cut from the steel tube. Coupons were tested according to Korean standard B0802. The yield stress of the steel tube was 49.6 (342 kg/cm^2) ksi, 56.1 ksi (387 kg/cm^2), and 52.9 ksi (365 kg/cm^2), and the modulus of elasticity ranged from 30595 (2110 t/cm^2) ksi to 33060 ksi (2280 t/cm^2). This study was for the investigation of the structural behavior of RCFT columns subjected to concentric force. The test parameters were the width to thickness ratio of steel tubes, and width-length ratio.

Concrete. The concrete mix was designed for compressive cylinder strength at 28 day with the Korean standard KS F 2404. The target strength of the concrete was 2.6 ksi (180 kg/cm2) and 6.5 ksi (450 kg/cm^2). The mix proportions for 2.6 ksi (180 kg/cm2) were 20.9 pcf (334 kg/cm^3) of cement, 11.6 pcf (86 kg/cm^3) of water, 50.1 pcf (802 kg/cm^3) of sand and 62.0 pcf (993 kg/cm^3) of coarse aggregate. The mix proportions for 6.5 ksi (450 kg/cm^2) were 33.1 pcf (530 kg/cm^3) of cement, 10.6 pcf (170 kg/cm^3) of water, 37.3 pcf (597 kg/cm^3) of sand and 68.0 pcf (1090 kg/cm^3) of coarse aggregate.

Length. The length of the columns was designed to avoid buckling. The length of the columns was 11.8 in. (300 mm). The support condition at the ends was designed as a hinge to minimize the effect of eccentricity. The axial force was transmitted by a spherical block which was installed on the bottom of the specimens.

<u>RCFT Columns 178-185</u>

General. As part of a test series that comprised of both circular CFT columns and rectangular columns, Kang et al. (2002) tested 8 circular CFT columns subjected to concentric load. The depth-to-thickness ratio (D/t) was 17.2, 27.3, 23.4, 31.3 and 43.5, respectively. The depths of the columns were 2 in. (5 mm), 3 in. (7.5 mm), and 4 in. (10 mm). Test parameters consisted of the steel shape (circular and square), the depth-thickness-ratio and in-filled concrete on the axial strength of stub column.

Steel. Eight types of steel plate were used (SPS400). Three coupons from each steel plate were taken and tested by the Korean standard B 0801. The yield stress of the steel tube was 40.2 ksi (2.8 t/cm^2), 40.6 ksi (2.8 t/cm^2), 49.7 ksi (3.4 t/cm^2), and 52.9 ksi (3.7 t/cm^2), respectively.

Concrete. The target strength of the concrete was 7.3 ksi (500 t/cm^2). Three cylinders were made from each batch, and tested in accordance with Korean standard F 2403 at 28 day. There were two types of concrete: normal Portland cement concrete and polymer cement concrete. The mixed proportions for normal concrete were 30.0 pcf (480 kg/m^3) of cement, 10.2 pcf (163 kg/m^3) of water, 52.9 pcf (848 kg/m^3) of fine aggregate and 50.9 pcf (815 kg/m^3) of coarse aggregate. The mixed proportions for polymer concrete were 5.6 pcf (90 kg/m^3) of polymer, 28.1 pcf (450 kg/m^3) of cement, 7.9 pcf (126 kg/m^3) of water, and 45.1 pcf (723 kg/m^3) of fine aggregate and 52.9 pcf (848 kg/m^3) of coarse aggregate. The concrete compressive strength was 7.9 ksi (542 kg/cm^2), and 6.8 ksi (467 kg/cm^2).

Length. The length of the columns was 5.9 in. (50 mm), 8.9 in. (75 mm), and 11.8 in. (100 mm). The length of the columns was three times that of the section depth.

<u>RCFT Columns 186-212</u>

General. Columns 186-212 were tested by Lee et al. (2002). The depth-to-thickness ratio (D/t) was 23.4, 31.3, and 43.5. The depths of the columns were 3 in. (75 mm), and

3.9 in. (100 mm). This paper presented the properties of structural behaviors for high strength CFT columns, such as kinds of concrete (Zeolite, fly ash, and silica fume), diameter to thickness ratio, and slenderness ratio.

Steel. Tensile coupons were manufactured in accordance with Korean standard B 0801. The yielding stress of the steel tube was 51.4 ksi (3.5 t/cm^2), 53.9 ksi (3.7 t/cm^2), and 54.3 ksi (3.74 t/cm^2). The elongation ratio ranged from 17.5 % to 20.8%, and the modulus of elasticity ranged from 29928 (2064 t/cm^2) ksi to 34409 ksi (2373 t/cm^2).

Concrete. Nine cylinders were manufactured from each batch, and tested in accordance with Korean standard F 2404 at 28 day. The target strength of the concrete was 8.7 ksi (600 kg/cm^2). The mixed proportions were 30.0 pcf (480 kg/m^3) of cement, 10.0 pcf (160 kg/m^3) of water, and 58.1 pcf (931 kg/m^3) of coarse aggregate. The concrete compressive strength was 8.03 ksi (554 kg/cm^2).

Length. The lengths of the columns were 10.8 in. (274.4 mm), 14.7 in. (374.4 mm), 15 in. (38.16 mm), 21.6 in. (548.8 mm), 29.5 in. (748.8 mm), 30 in. (763.2 mm), 32.4 in. (823.2 mm), 44.2 in. (1123.2., and 45.1 in. (1144.8 mm).

RCFT Columns 213-222

General. As part of a test series that comprised both rectangular CFT columns and beam-columns, Seo et al. (2002) tested 10 rectangular CFT columns. The depth-to-thickness ratio (D/t) was 42.1. The depths of the columns were 4.91 in. (124.9 mm), and 4.93 in. (125.1 mm). The nominal section was 4.9 in. (125 mm) x 4.9 in. (125 mm) x 0.13 in. (3.2 mm) (SS400). Experimental parameters were the ratio of buckling length to the depth of the section. The experimental parameters investigated were the strength capacity, ductility and flexural stiffness of the concrete filled in tube beam-columns for both compact and slender HSS sections.

Steel. Tensile coupons were cut from the steel tube and were manufactured according to the JIS (Japanese Industrial Standard). The yield stress of the steel tube was 63.2 ksi (4.4 t/cm^2), 64.5 ksi (4.5 t/cm^2), and 65.7 (4.5 t/cm^2) ksi.

Concrete. The target strength of the concrete was 8.5 ksi (58.8 N/mm^2). The compressive strength of the concrete at 28 days ranged from 8.9 ksi (62.2 N/mm^2) to 9.8 ksi (68.9 N/mm^2). The mix proportions were 32.2 pcf (516 kg/m^3) of cement, 10.6 pcf (170 kg/m^3) of water, 52.4 pcf (840 kg/m^3) of fine aggregate, and 53.2 pcf (852 kg/m^3) of coarse aggregate.

Length. The lengths of the columns were 19.7 in. (500 mm), 39.4 in. (1000 mm), 59.1 in. (1500 mm), 88.6 in. (2250 mm), and 118.1 in. (3000 mm). An end plate of thickness 1.3 in. (32 mm) was welded at both ends.

3.2.6 RCFT Beam-Column

RCFT Beam-Column 83-86

General. The series of tests included two columns bent about the minor axis, two columns bent about major axis, and four columns subjected to biaxial bending (Wang and Moor, 1997). The end plates were manufactured and fixed to both ends of each column to form a simply supported column under double curvature bending. Test parameters were the bending axis and effective length.

Steel. A hot rolled 4.72 in. (120 mm) x 3.15 in. (80 mm) x 0.248 (6.3 mm) rectangular section was selected to give a relative slenderness of about 1.0. The average measured steel yield stress of 53.7 ksi (370 N/mm^2) was used for all columns.

Concrete. The concrete strength of 8.7 ksi (60 N/mm^2) qualified as a C50/60 grade. Therefore, a characteristic strength of 6.25 ksi (50 N/mm^2)was used with the Eurocode 4 method.

Length. A column length of 126.1 in. (3200 mm) and 157.5 in. (4000 mm) was chosen because this is the typical of the story height in a multistory building.

RCFT Beam-Column 87-103

General. As part of a test series that comprised both columns and beam-columns, Matsui et al. (1997) tested 17 rectangular CFT beam-columns. The depth-to-thickness ratio of the steel plate element was 35.1. The design method according to the AIJ standards for concrete filled steel tubular structures is summarized. A modified AIJ method has been proposed by the authors. The difference between the AIJ and the modified AIJ method is only in the strength of the concrete column. In modified AIJ method, high accurate concrete column strength based on a numerical analysis is proposed.

Steel. The material of the steel portion was mild steel (STKP400 (square), Japanese Industrial Standards). Tests showed that the square tube had the yield strength of 59.7 ksi (4.2 t/cm^2).

Concrete. The average compressive strength of concrete for square columns was 5.93 ksi (325 kg/cm^2).

Length. The The effective length of columns ranged from 23.62 in. (600 mm) to 177.3 in. (4956 mm).

<u>RCFT Beam-Columns 104-115</u>

General. Beam-columns 104-115 were tested by Hardika and Gardner (2004). The depth of the columns was 8.0 in. (203 mm). The depth-to-thickness ratios (D/t) were 22.2, and 22.7. The ratio of the steel and total area ranged from 8.5% to 16.9%. The experimental variables were the strength, ductility and flexural stiffness of the concrete filled in tube beam-columns for both compact and slender HSS sections.

Steel. The minimum yield strength of the steel tube in accordance with ASTM A 500, Grade requirements was, 50 ksi (345 MPa). The yield stress from coupons tests was 54.8 ksi (378 MPa), 56.6 ksi (390 MPa), 57.0 ksi (393 MPa), and 59.7 ksi (411 MPa).

Concrete. The target design strengths were 5.8 ksi (40 MPa) for normal strength concrete and 13.0 ksi (90 MPa) for high strength concrete. Both normal and high strength concrete were ordered from a local ready mixture supplier. The compressive strength of the concrete at test day was 6.4 ksi (44.4 MPa) for normal strength concrete, and 12.0 ksi (82.9 MPa) to 14.4 ksi (99.1 MPa) for high strength concrete.

Length. Column specimen length was determined by the length of the MTS hydraulic actuators and embedded length of the column in foundation. While the total length of the column was 75 in. (1900 mm), the effective columns length was 71 in. (1800 mm) because the embedded length was 15 in. (381 mm) and the loads were applied through the loading beam 11 in. (280 mm) above the top of the column.

RCFT Beam-Columns 116-127

General. As part of a test series that comprised both rectangular CFT columns and beam-columns, Han and Yao (2002) tested 12 rectangular CFT beam-columns. The depth size of the columns was 7.7 in. (195 mm), 5.1 in. (130 mm). The depth-to-thickness ratios (D/t) were 49.1. The ratio of the steel and total area was 6.7%.

Steel. The steel tubes were all manufactured from mild steel sheet, with four plates were cut from the sheet, tack welded into a rectangular shape and then welded with a single bevel butt weld at the corners. Strips of the steel tubes were tested in tension in accordance with Chinese standard GB2975. Three coupons were taken from each face of the steel tube. From these tests, the average yield strength of the tube was found to be 49.3 ksi (340.1 MPa) and the modulus of elasticity was about 30015 ksi (207000 MPa). The end of steel tubes were cut and machined to the required length. The insides of the tubes were wire brushed to remove rust and loose debris present. The deposits of grease and oil, if any, were cleaned away. Each tube was welded to a rectangular steel base plate 10 mm thick. The specimen was placed upright to air-dry until testing occurred. The present study is an attempt to study the influence of concrete compaction on the strength of concrete filled steel RHS columns. Thirty five concrete filled steel RHS columns were tested to investigate the influence of concrete compaction methods compared to the member capacities of the composite columns. The main parameters which varied in the tests were the column section depth-to-width ratio, tube depth to thickness ratio, load eccentricity, and column slenderness.

Concrete. The concrete mix was designed for compressive cube strength at 28 days with the Chinese standard GBJ81-85, the average value being 3.7 ksi (253 MPa) The mix proportions were 25.2 pcf (403 kg/m^3) of cement, 9.6 pcf (153 kg/m^3) of water, 35.0 pcf (561 kg/m^3) of sand and 80.1 pcf (1283 kg/m^3) of coarse aggregate. The average cube strength at the time of the test was 3.4 ksi (23.1 MPa). In all thee concrete mixes, the fine aggregate used was silica-base, the coarse aggregate was carbonate stone. The specimen tests allowed the different conditions likely to arise in the manufacture of concrete: cured, well compacted with a poker vibrator and well compacted by hand. During curing, a very small amount of longitudinal shrinkage of 0.02 in. (4 mm)-0.03 in. (7 mm) or so occurred at the top of the column.

Length. The lengths of columns were 30.7 in. (780 mm) and 92.1 in. (2340 mm). Both the end plate and the top plate were made of very hard and very high strength steel.

RCFT Beam-Columns 128-134

General. As part of a test series that comprised both rectangular CFT columns and beam-columns, Uy (2000) tested 9 rectangular CFT beam-columns. The depths of the columns were 4.3 in. (110 mm), 6.3 in. (160 mm), and 8.3 in. (210 mm). This study included an extensive set of experiments on high strength steel box columns filled with concrete and a numerical model which is presented and calibrated. The main parameters were tube width, depth to thickness ratio, concrete compressive strength, yielding stress of steel tube, eccentricity, and local buckling.

Steel. Residual stress measurements were conducted using a combination of both electric strain gauges and mechanical strain gauges across the width of the plates. The yield stress of the steel tube was 108.8 ksi (750 MPa).

Concrete. To determine the average compressive strength of the concrete a series of standard cylinders were crushed throughout the testing period. The average of compressive strength was given for different ages of testing. The reported compressive strengths of the concrete were 4.35 ksi (30 MPa) and 4.64 ksi (32 MPa).

Length. To ascertain a uniform loading surface, columns were cast in place with plates with plaster at either end. The eccentrically loaded columns were loaded using a knife-edge at both the top and bottom of the column. The length of the columns was 118.1 in. (3000 mm).

<u>RCFT Beam-Columns 135-145</u>

General. As part of a test series that comprised both rectangular CFT columns and beam-columns, Seo, and Chung (2002) tested 11 rectangular CFT beam-columns. The depth-to-thickness ratio (D/t) was 39.1. The depth of the columns was 4.9 in. (125 mm). Test parameters were the bucking length-section ratio and the eccentricity of the applied compressive load.

Steel. To determine the yielding stress and material properties of steel tube, tensile coupon tests and stub column tests were conducted. Tensile coupons were manufactured in accordance with JIS Z2201 (Japanese Industrial standard). The yield stress of the steel tube was 65.5 ksi (4.5 t/cm^2). The elongation ratio ranged from 17.5 % to 20.8%.

Concrete. The water-cement ratio was 26%. The mix proportions were 32.5 pcf (520 kg/m^3) of cement, 10.6 pcf (714 kg/m^3) of water, 54.24 pcf (869 kg/m^3) of coarse aggregate and 44.57 pcf (714 kg/m^3) of fine aggregate. Design concrete compressive strength was 15.33 ksi (1057 kg/cm^2). The mean concrete compression strength at 28 day was 13.0 ksi (898 kg/cm^2). The mean concrete compression strength at test day was 13.9 ksi (960 kg/cm^2). To get high workability, silica fume was added.

Length. The length of the columns were 19.7 in. (500 mm), 39.4 in. (1000 mm), 59.1 in. (1500 mm), 88.6 in. (2250 mm), 118.1 in. (3000 mm) and 147.6 in. (3750 mm). Support condition of the columns was hinge

RCFT Beam-Columns 146-194

General. As part of a test series that comprised both rectangular rectangular CFT columns and beam-columns, Seo et al. (2002) tested 49 rectangular CFT beam-columns. The depth-to-thickness ratio (D/t) was 42.1. The depths of the columns were 4.91 in. (124.9 mm), and 4.93 in. (125.1 mm). The nominal section was 4.9 in. (125 mm) x 4.9 in. (125 mm) x 0.13 in. (3.2 mm) (SS400). Experimental parameters were the ratio of buckling length to the depth of the section. The experimental parameters investigated were the strength, ductility and flexural stiffness of the concrete filled in tube beam-columns for both compact and slender HSS sections.

Steel. Tensile coupons were cut from the steel tube were manufactured according to the JIS (Japanese Industrial Standard). The yield stress of the steel tube was 63.2 ksi (4.36 t/cm^2), 64.5 ksi (4.45 t/cm^2), and 65.7 (4.53 t/cm^2) ksi.

Concrete. The target strength of the concrete was 8.5 ksi (58.8 N/mm^2). The compressive strength of the concrete at 28 days ranged from 8.9 ksi (62.2 N/mm^2) to 9.79 ksi (68.9 N/mm^2). The mix proportions were 32.2 pcf (516 kg/m^3) of cement, 10.6 pcf (170 kg/m^3) of water, 52.4 pcf (840 kg/m^3) of fine aggregate, and 53.2 pcf (852 kg/m^3) of coarse aggregate.

Length. The length of the columns was 19.7 in. (500 mm), 39.4 in. (1000 mm), 59.1 in. (1500 mm), 88.6 in. (2250 mm), 118.1 in. (3000 mm) and 147.6 in. (3750 mm). An end plate of thickness 1.3 in. (32 mm) was welded at both ends.

CHAPTER Ⅳ

ANALYSIS AND RESULTS

The column databases described in the previous chapter were assembled without a critical assessment of the validity of the tests for calibration purposes. The first step in selecting the tests to be used for the development of new design equations was to eliminate from the database specimens that for one reason or another were deemed to be outside the scope of the work. Only tests that clearly fell outside the parameters were eliminated, and every effort was made at retaining as many tests as possible. This approach is significantly different from calibration efforts such as those carried out for the Eurocode, where only small subsets consisting of very well-documented tests were used. Both approaches are valid, but the choice made here was based on the assumption that many or the problems encountered in testing (accidental load eccentricities and the effects of friction at the ends) may reflect actual imperfections in practice. Because much of the effort in this work is geared towards comparing different specifications, the effect of any outlier data is assumed to be small in the comparisons.

The most important categories that were separated at this stage were:

- Specimens which did not achieve their ultimate strength due to well-documented problems during testing. These were the only tests that were completely eliminated from further analysis.

- Tests in which the specimens were subjected to cyclic loading, as these are often designed to be shear critical and subjected to double curvature. These specimens will be examined in a future work as part of an assessment of the provisions in Part II of the AISC Seismic Provisions.

Specimens in the following four categories were also removed and analyzed separately:

- Tests subjected to biaxial bending.
- Tests subjected to unequal end moments.
- Tests that did not meet applicable local buckling criteria.
- Tests containing lightweight concrete.

To compare the ultimate strength values predicted by the 1999 AISC, 2005 AISC and Eurocode 4 provisions, the ratio of the experimental axial capacity to the predicted axial capacity was used. This choice was driven primarily by the need to calibrate to axially-loaded columns first and then to beam-column strengths. No attempt was made to optimize the proposed 2005 AISC design provisions to both the column and beam-column data simultaneously. The comparisons were made both with and without resistance factors, as these vary considerably. In addition, some comparisons were also made of the slenderness parameters used by each specification, as the values of these parameters (α and λ) are defined differently and thus is impossible to compare columns with the same slenderness directly.

This chapter begins with a description of the limiting values found in the database for several parameters of interest. These include values and distributions of yield stress, concrete compressive strength, structural steel ratio, e/D ratio for beam-columns, and D/t ratio for circular concrete filled tubes and B/t ratio for rectangular concrete filled tubes.

4. 1 WORKING DATA SUMMARY

4.1.1 SRC Columns

The preliminary database for encased (SRC) columns consisted of 119 SRC columns. From that data set, 27 SRC columns were separated as they utilized lightweight concrete (Stevens, 1965; Janss and Pirapez 1974). In addition, three specimens were removed because of problems during testing (Janss and Anslijn 1974). As a result, 89 SRC columns were used for the final analyses. The subset thus obtained is labeled the reduced database and is the data set used for all comparisons.

As shown in Figure 4-1, which illustrates the distribution of yield strengths in the tests, the maximum and minimum yield stress were 72.7 ksi and 32.4 ksi, respectively. The figure indicates relatively few column tests with high strength steels, and even fewer beam-columns. The distribution of concrete compressive strength for this data set is shown in Figure 4-2. The compressive strength ranges from 9.52 ksi to 1.84 ksi. The structural steel ratio, shown in Figure 4-3, ranged from 2.7% to 12.9%. Both Figures 4-2 and 4-3 show a more uniform distribution than Figure 4-1. Figure 4-4 shows a 3D plot of the yield stress, concrete compressive strength and reinforcement ratio for columns. This plot emphasizes the large gaps in the database from 6 to 8 ksi in compressive strength of the concrete, around 60 ksi of yield stress and over 0.13 of structural steel ratio. Figures 4-5, 4-6 and 4-7 show scatter plots of the data for AISC 1999, AISC 2005 and Eurocode 4 versus the slenderness parameter.

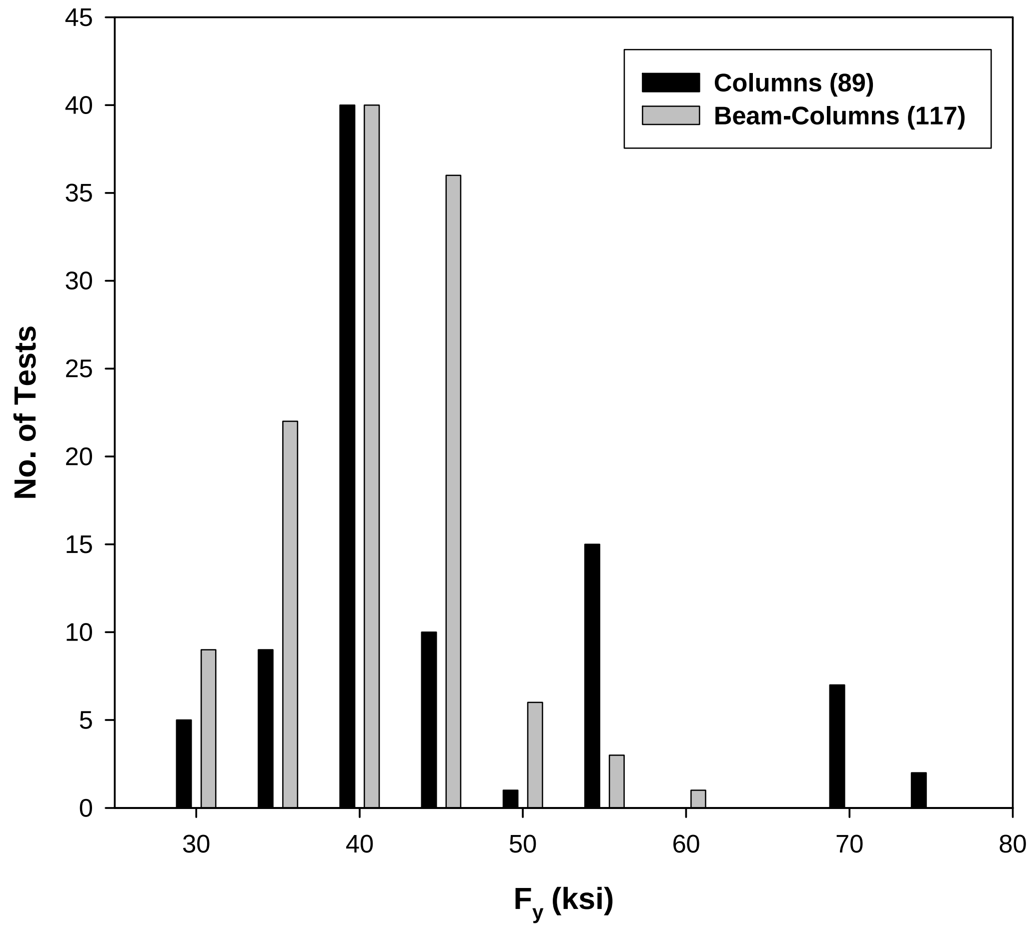


Figure 4-1 Frequency distribution of F_y for the reduced SRC database

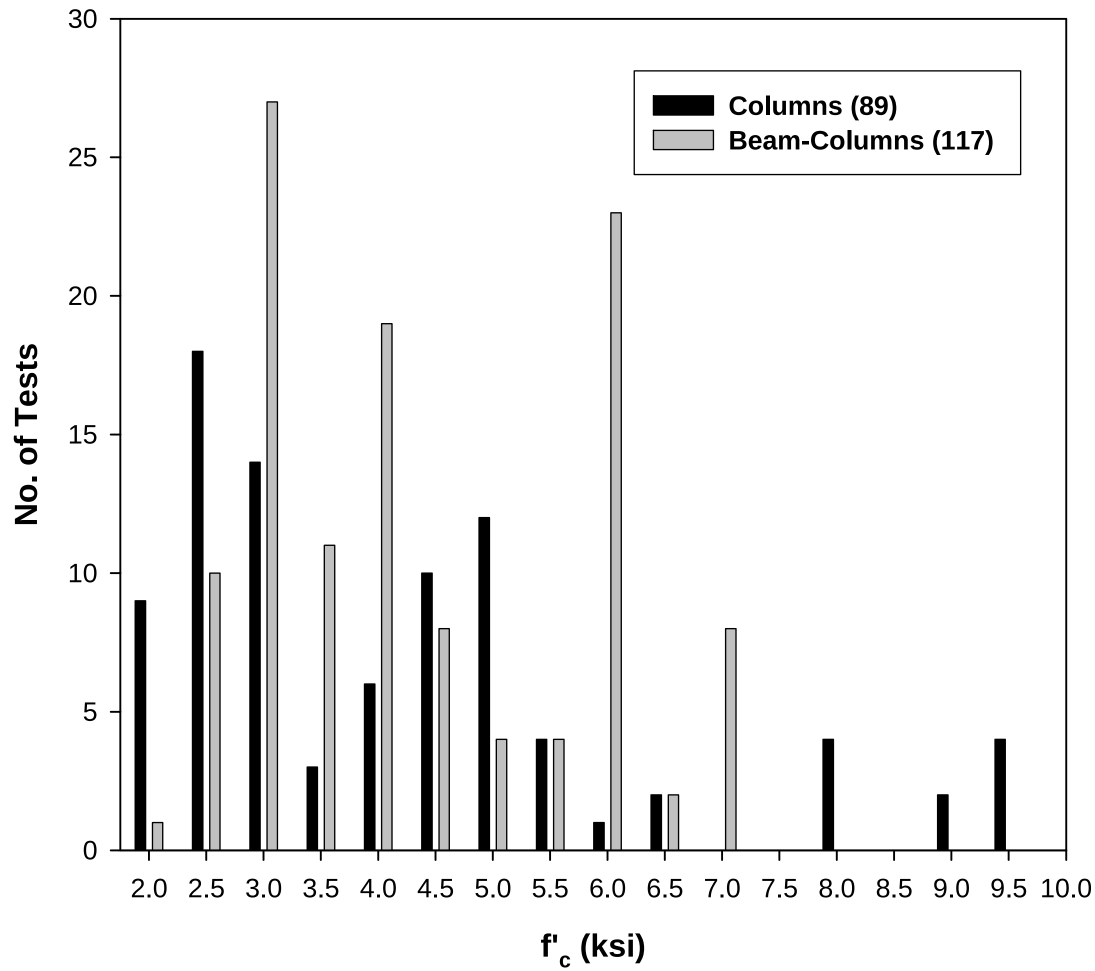


Figure 4-2 Frequency distribution of f'$_c$ for the reduced SRC database

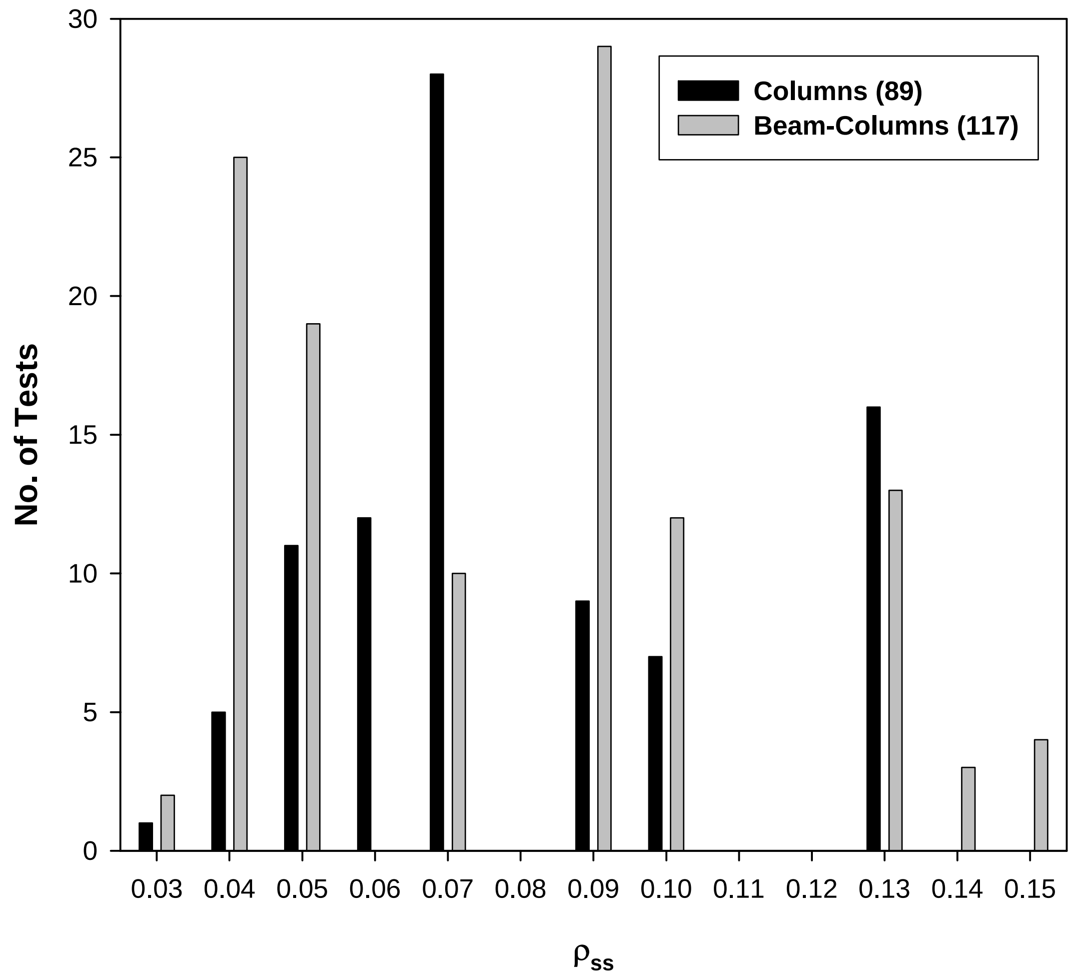


Figure 4-3 Frequency distribution of ρ_{ss} for the reduced SRC database

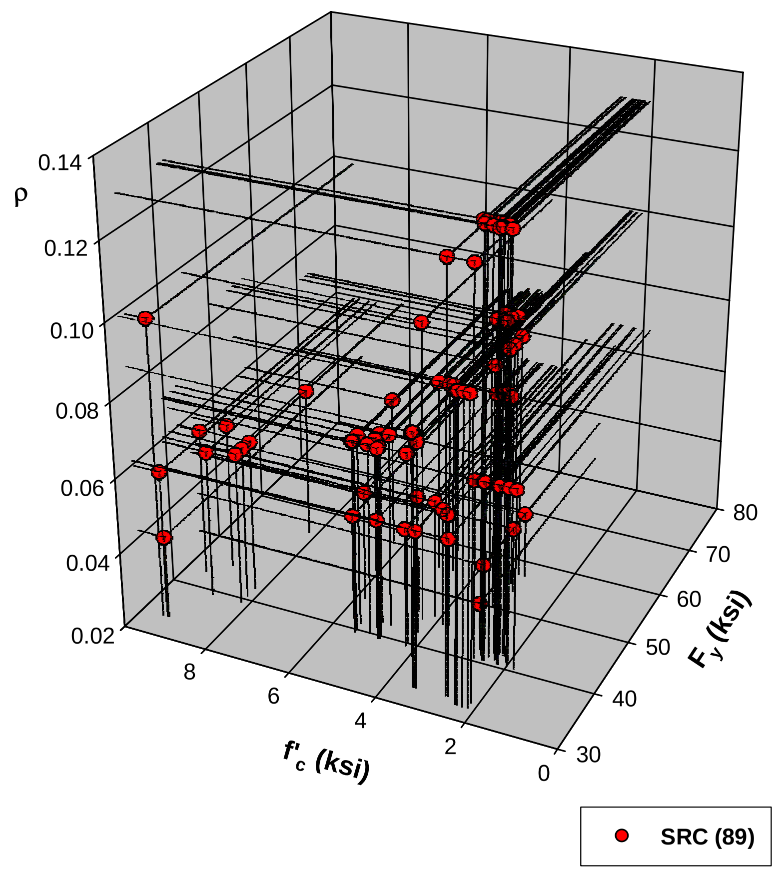


Figure 4-4 Frequency distribution of F_y, f'_c and ρ_{ss} for the reduced SRC database

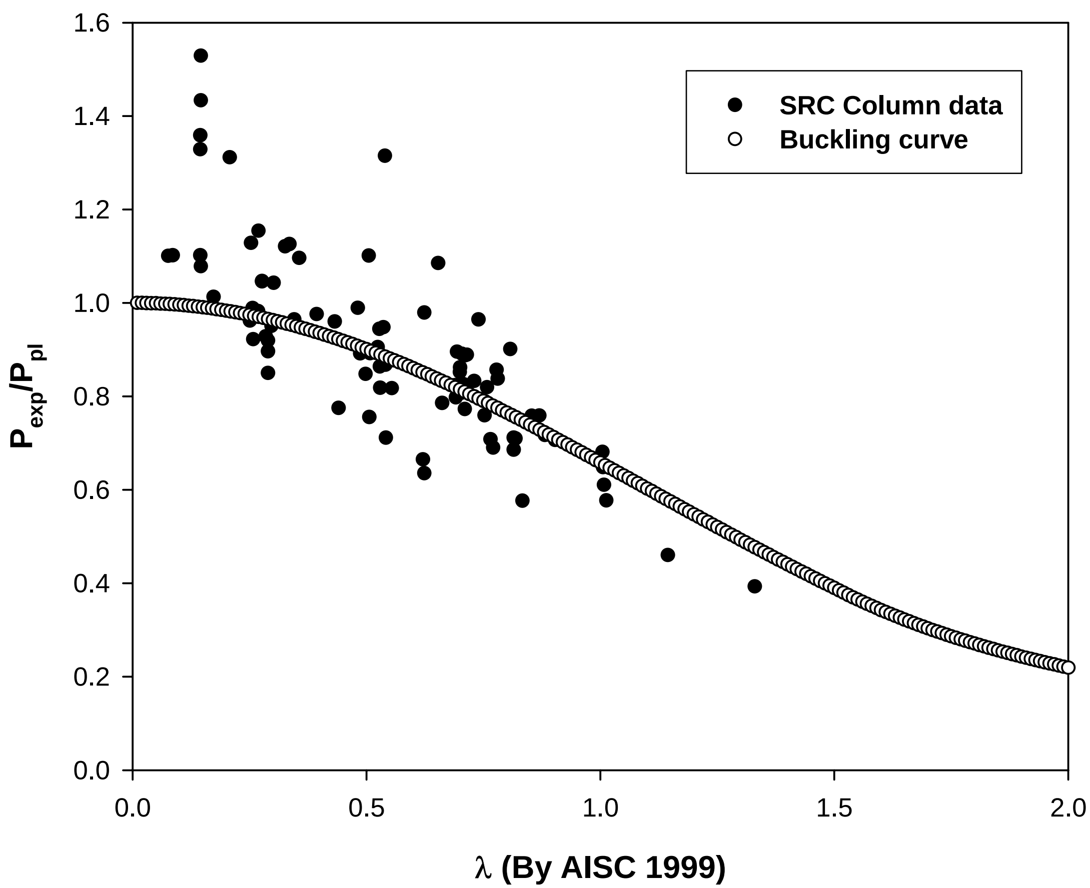


Figure 4-5 P_{exp}/P_{pl} with AISC buckling curve for SRC columns by AISC 1999

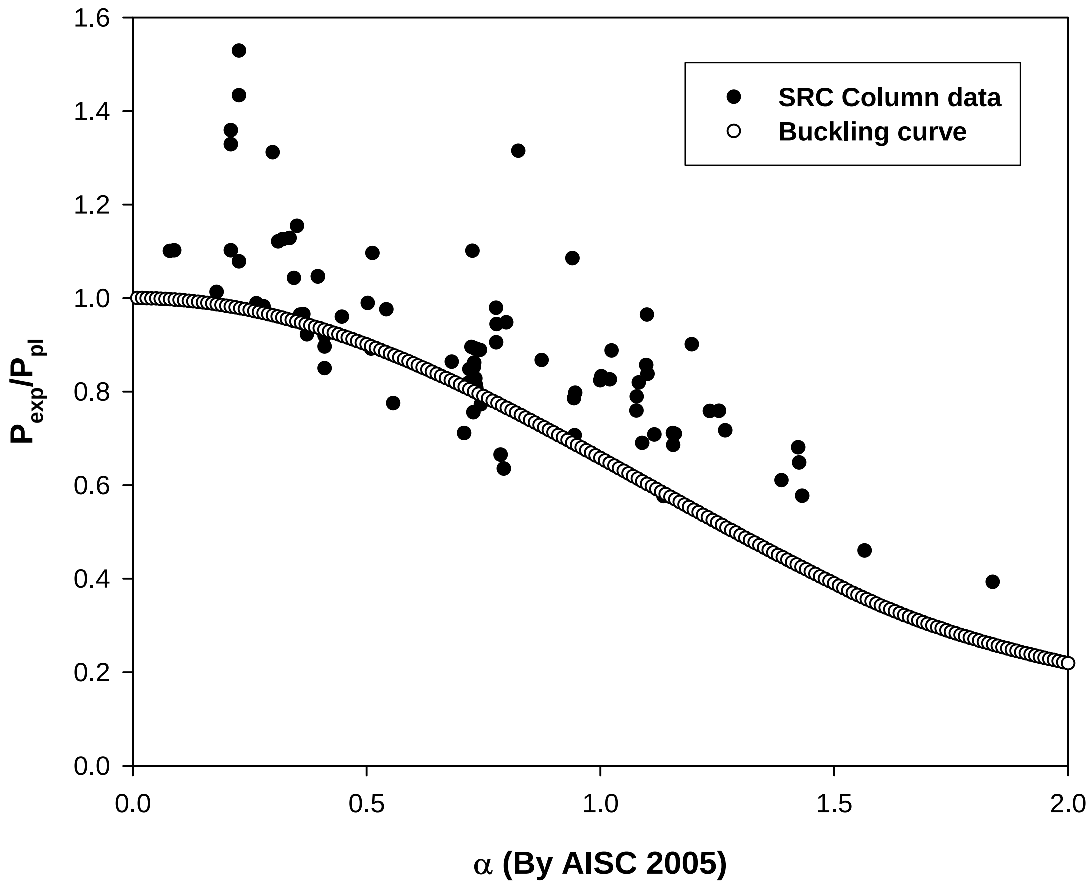


Figure 4-6 P_{exp}/P_{pl} with AISC buckling curve for SRC columns by AISC 2005

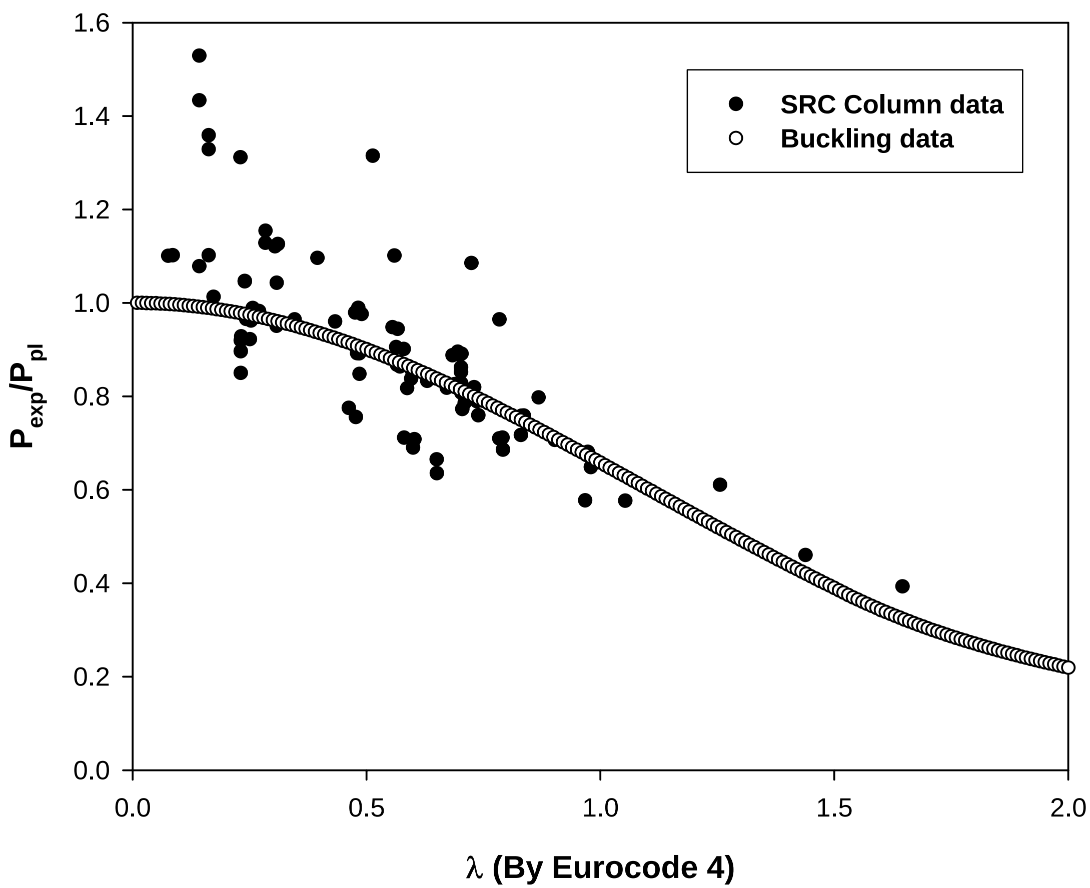


Figure 4-7 P_{exp}/P_{pl} with AISC buckling curve for SRC columns by Eurocode 4

Table 4-1 Comparison of column strengths and α/λ ratios

Type		No. of Tests	Mean	Standard Deviation	COV	Mean Design Axial Load(AISC) 2005/1999 (COV)
Encased columns (SRC)	AISC 1999	89	1.22	0.19	0.16	1.11 (0.15)
	AISC 2005		1.18	0.20	0.17	
	Eurocode 4		1.09	0.14	0.13	
	α/λ (AISC 2005/ AISC 1999)		1.32	0.18	0.13	
	α/λ (AISC 2005/ Eurocode 4)		1.34	0.26	0.19	
Encased columns (SRC) (Light weight concrete)	AISC 1999	27	1.19	0.17	0.14	
	AISC 2005		1.15	0.29	0.25	
	Eurocode 4		1.01	0.17	0.17	

Table 4-1 shows a summary of the comparisons between the three specifications for encased columns. The mean experimental to calculated unfactored axial capacity by the AISC 1999 method was 1.22, with a standard deviation of 0.19 and coefficient of variation of 0.16. The maximum and minimum ratios were 1.94 and 0.86, respectively. When the resistance factor of 0.85 was considered, the mean ratio, standard deviation and coefficient of variation changed to 1.43, 0.23 and 0.16, respectively. By the AISC 2005 method, the mean ratio was 1.18, with a standard deviation of 0.2 and coefficient of variation of 0.17. The maximum and minimum ratios were 1.75 and 0.83, respectively. When the resistance factor of 0.75 was added, the mean changed to 1.48 with a standard deviation of 0.26.

The unfactored mean of 1.22 by the AISC 1999 method was larger than that of 1.18 by the AISC 2005 specification, but the design values are only somewhat more conservative by the 1999 procedure because of the corresponding large difference in resistance factors. This improved mean of the AISC 2005 method results primarily from a better fit of the experimental values to the new definition of the slenderness parameter. This, in turn, results in a shift of the buckling curve.

mean by the Eurocode was 1.09 with a standard deviation of 0.14 and a coefficient of variation of 0.13. When a partial safety factor of 1.1 for the structural steel, of 1.5 for the concrete, and of 1.15 for the reinforcing steel are used, the mean was 1.37 with a standard deviation of 0.18 and a coefficient of variation of 0.13. For SRC columns and this large data set, the Eurocode has a very better mean value and standard deviations compared with both the 1999 AISC and 2005 AISC. The relatively large standard deviation for the latter is an indication of the scatter and irregular distribution of the data. Thus, the Eurocode predicts strength very well for the smaller database in which it is based, and works well for other kinds of composite columns.

For the subset of data with lightweight crete shown in Table 4-1, the mean values are lower than those for normal weight crete. As a result, the predicted axial capacity for lightweight concrete columns is less conservative than that for the normal weight concrete columns. Figure 4-5, Figure 4-6 and Figure 4-7 does not include lightweight concrete column data.

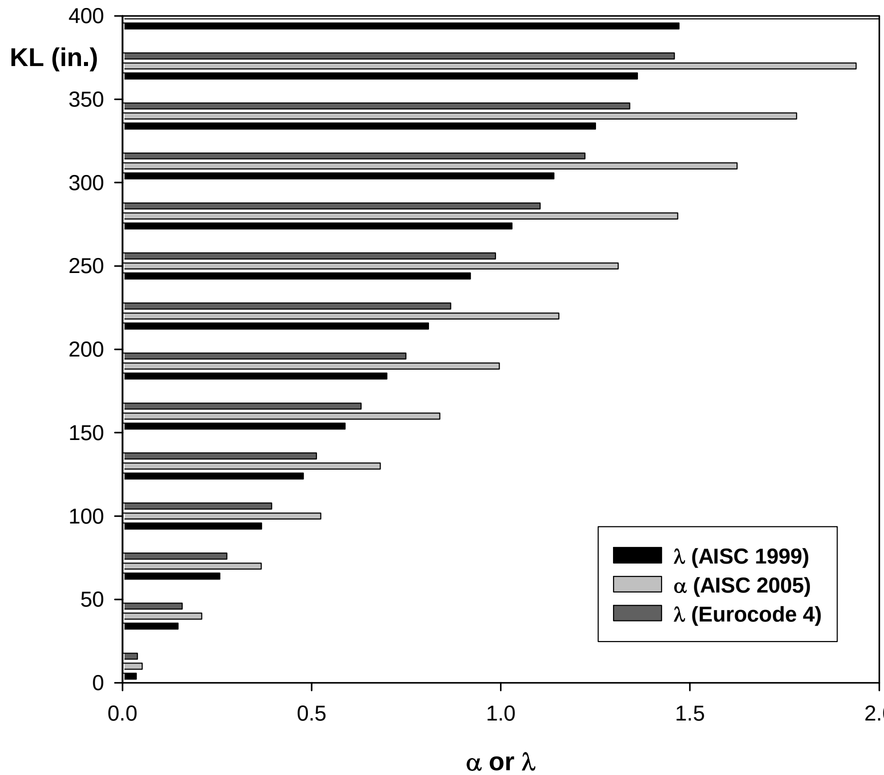


Figure 4-8 Comparison of slenderness ratio for a typical column in the SRC database (Astaneh-Asl, Chen, and Moehle, 1992)

The mean of α/λ is 1.32 for AISC 2005/AISC 1999 and 1.34 for AISC 2005/ Eurocode 4. The difference between the two slenderness parameters increases with increasing column slenderness. Figure 4-8 shows a comparison of slenderness ratios for 75th column in the database of SRC columns given an effective length. Interestingly, when the AISC 2005 and Eurocode 4 are compared, the value of α is always larger than that of λ for SRC columns. This is due to the fact that the effective EI by Eurocode 4 is larger than that by AISC 2005. Note that the difference in the values of the slenderness parameter does not necessarily translate into a similar difference in load carrying capacity.

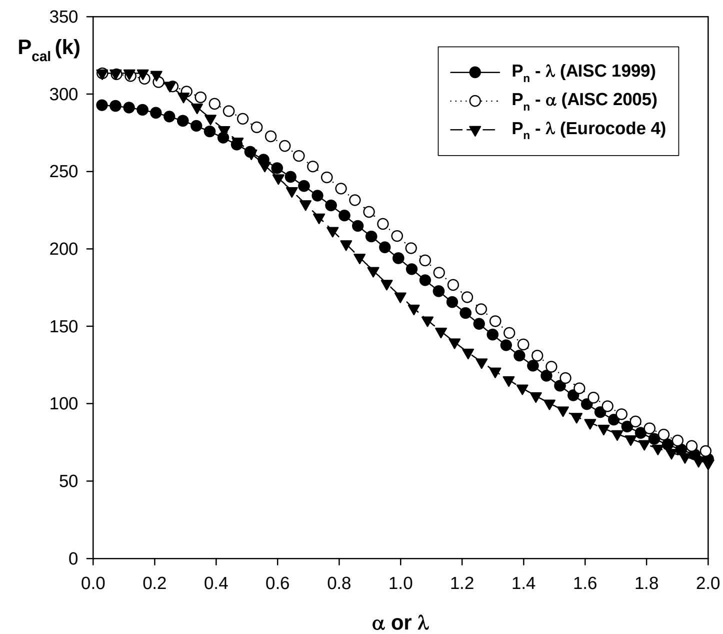


Figure 4-9 al capacity vs. slenderness for a typical umn in SRC database (Stevens, 1965)

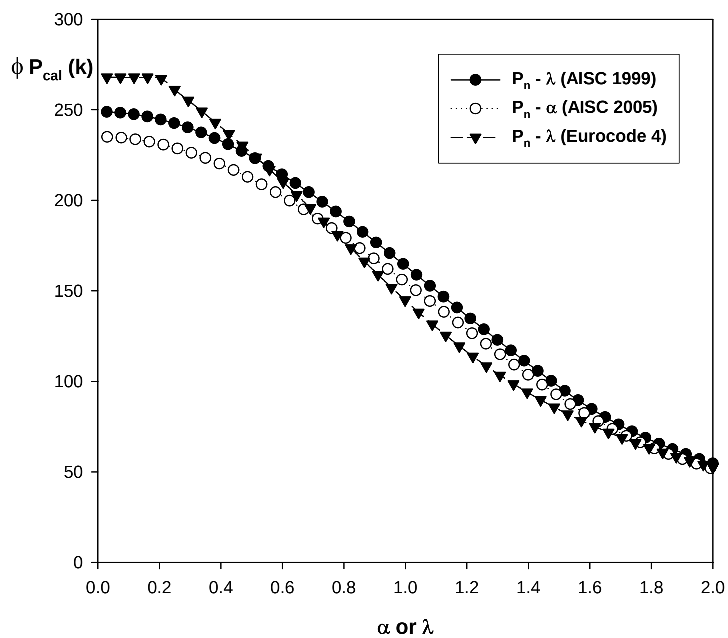


Figure 4-10 Axial capacity vs. slenderness for a typical umn in SRC database (Stevens, 1965)

Figures 4-9 and 4-10 show two examples of the strength vs. slenderness curves for a typical specimen in the database (39th column, Stevens, 1965). As shown Figure 4-9, the P_{cal} value by AISC 2005 is larger than that from the AISC 2005 as the slenderness decreases if compared without resistance factors. However, the ϕP_{cal} value given by AISC 2005 is less than that from the AISC 1999 if the design capacity is compared. In contrast, the P_{cal} value given by Eurocode 4 is usually larger at a low slenderness ratio when both resistance factors and no resistance factors are used.

4.1.2 SRC Beam-Columns

There were a total 136 SRC beam-columns in the initial database. Twelve SRC beam-columns were eliminated in database because of biaxial bending (Virdi and Dowling, 1972), four specimens were eliminated because of unequal end moment (Johnson and May, 1978 ; Roik and Schwalbenhofer, 1989) and three tests were eliminated because of excessive eccentricity (A. Mirza, V. Hyttinen, and E. Hyttinen, 1997). Thus, 117 SRC beam-columns were used for analysis and included in the database. The distribution of parameters is shown in Figure 4-1 through 4-3. As shown in these figures, yield stress varied between 32.3 ksi and 58 ksi., while the maximum compressive strength was 6.8 ksi and the minimum was 1.8 ksi. The structural steel ratio ranged from 2.7% to 14.6%. Figure 4.11 shows a 3D plot of the yield stress, concrete compressive strength and reinforcement ratio for beam-columns. This plot emphasizes the large gaps in the database around 7 ksi in compressive strength of the concrete, more than 60 ksi of yield stress and around 0.11 of structural steel ratio. Figure 4-13, 4-14 and 4-15 show a scatter plot of the data by AISC 1999, AISC 2005 and Eurocode 4 versus the slenderness parameter. The values are normalized by P_{pl}, which is the plastic axial load capacity.

SRC beam-column data by AISC 2005 is evenly distributed. The distribution of e/D (eccentricity to depth of section) is shown in Figure 4-12. Values for e/D ranged from 0.03 to 1.55.

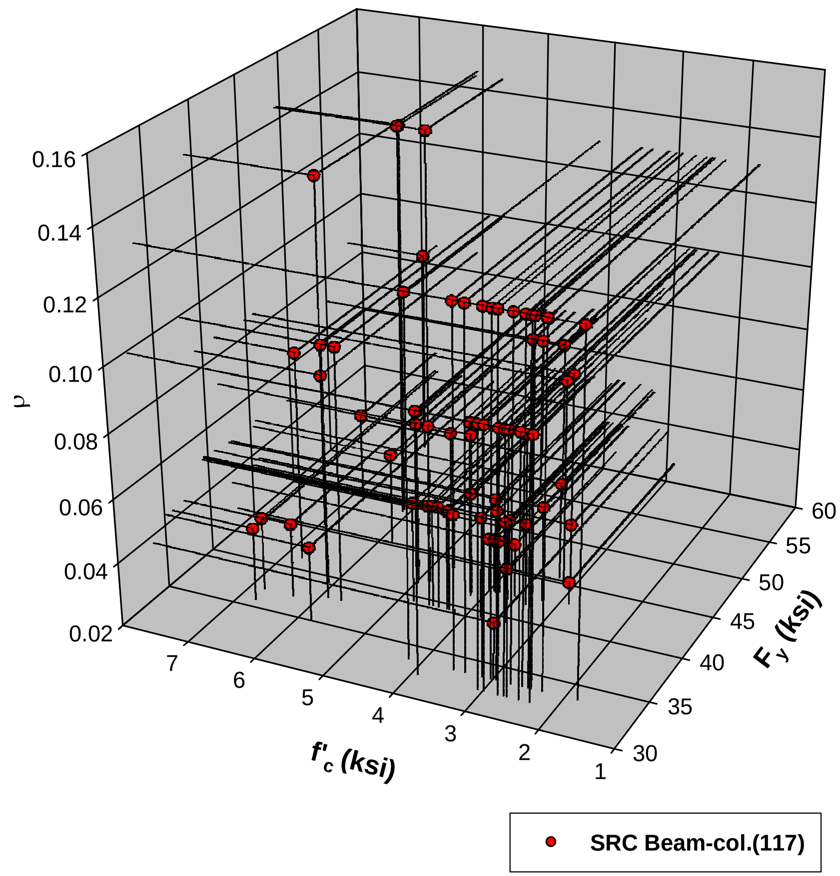


Figure 4-11 Frequency distribution of F_y, f'_c and ρ_{ss} for the reduced SRC beam-columns database

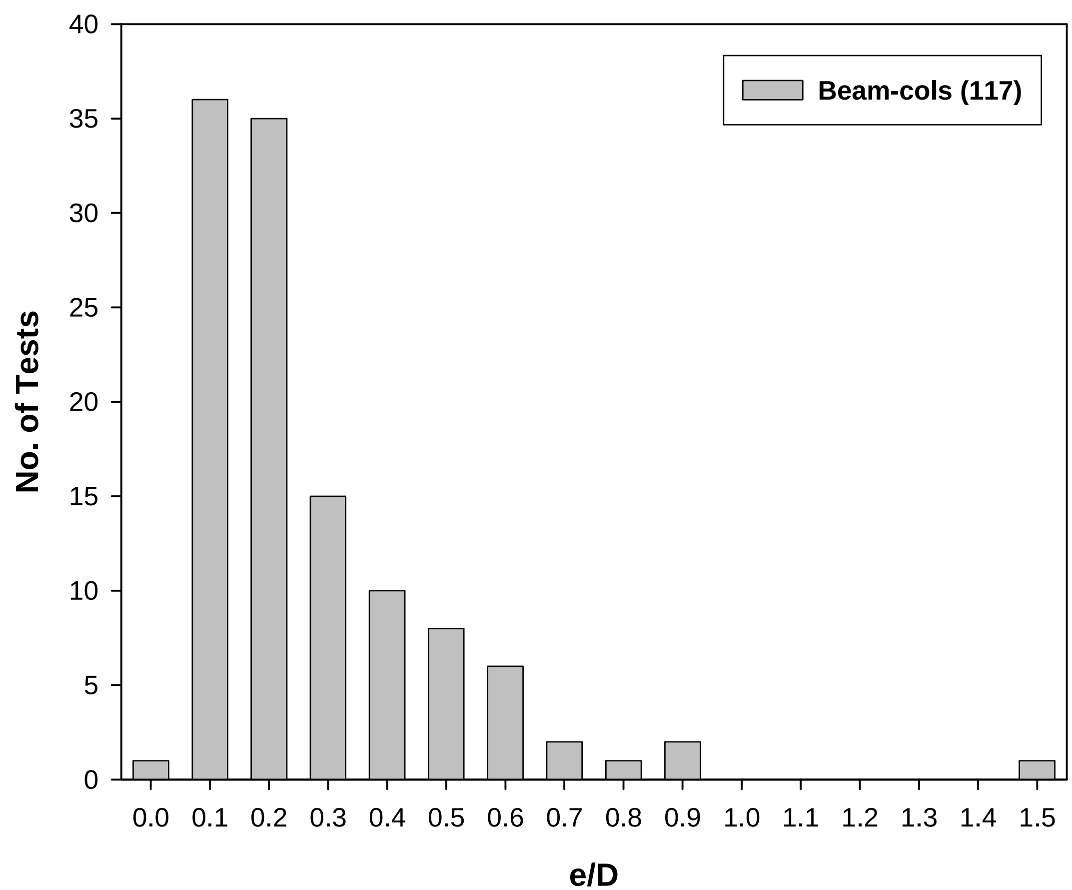


Figure 4-12 Frequency distribution of e/D for the reduced SRC beam-column database

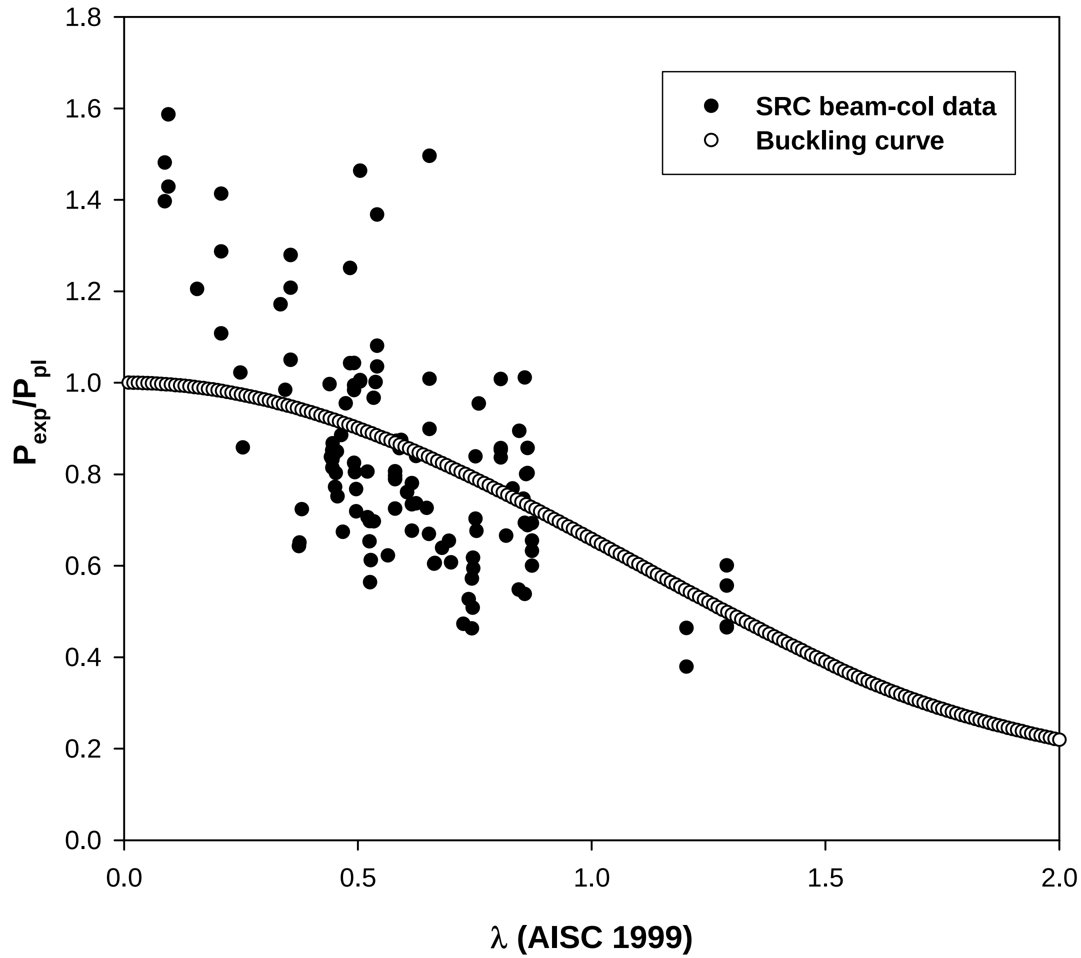


Figure 4-13 P_{exp}/P_{pl} with AISC buckling curve for SRC beam-columns by AISC 1999

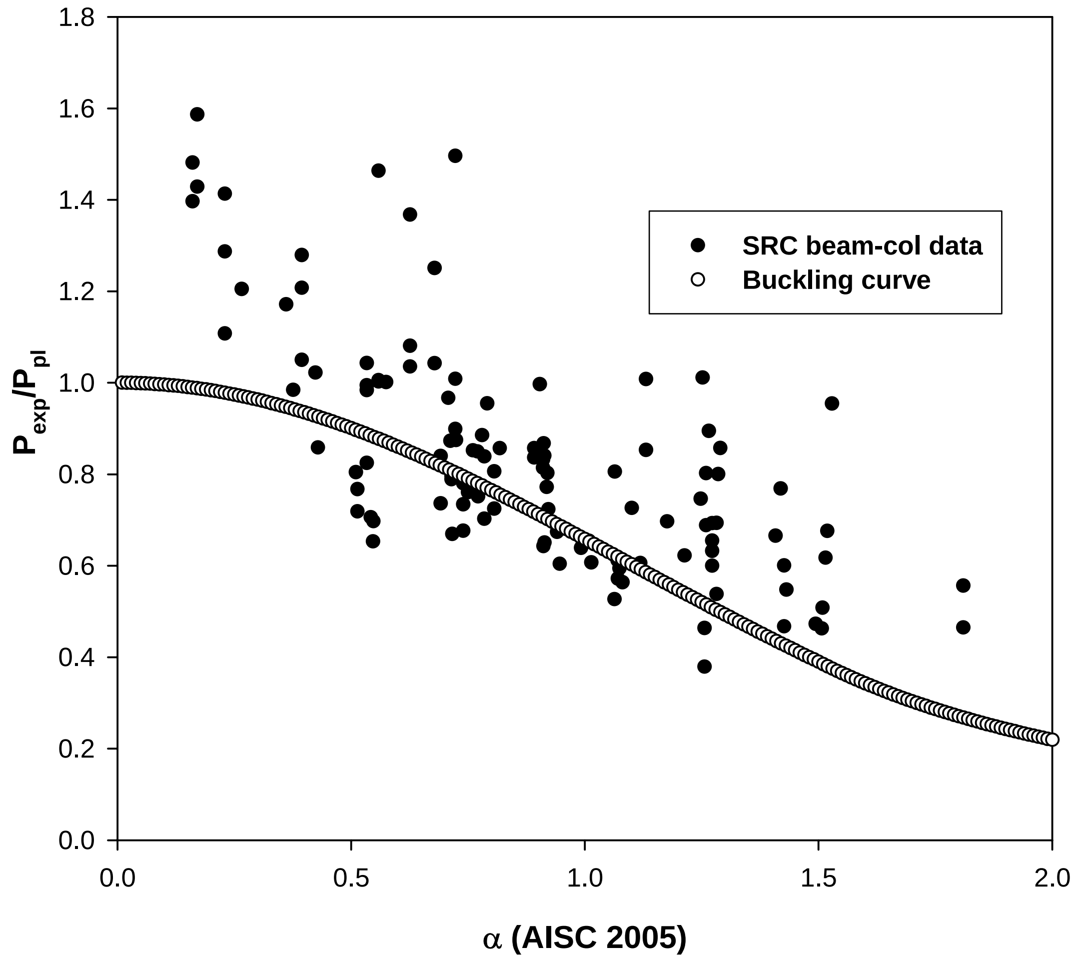


Figure 4-14 P_{exp}/P_{pl} with AISC buckling curve for SRC beam-columns by AISC 2005

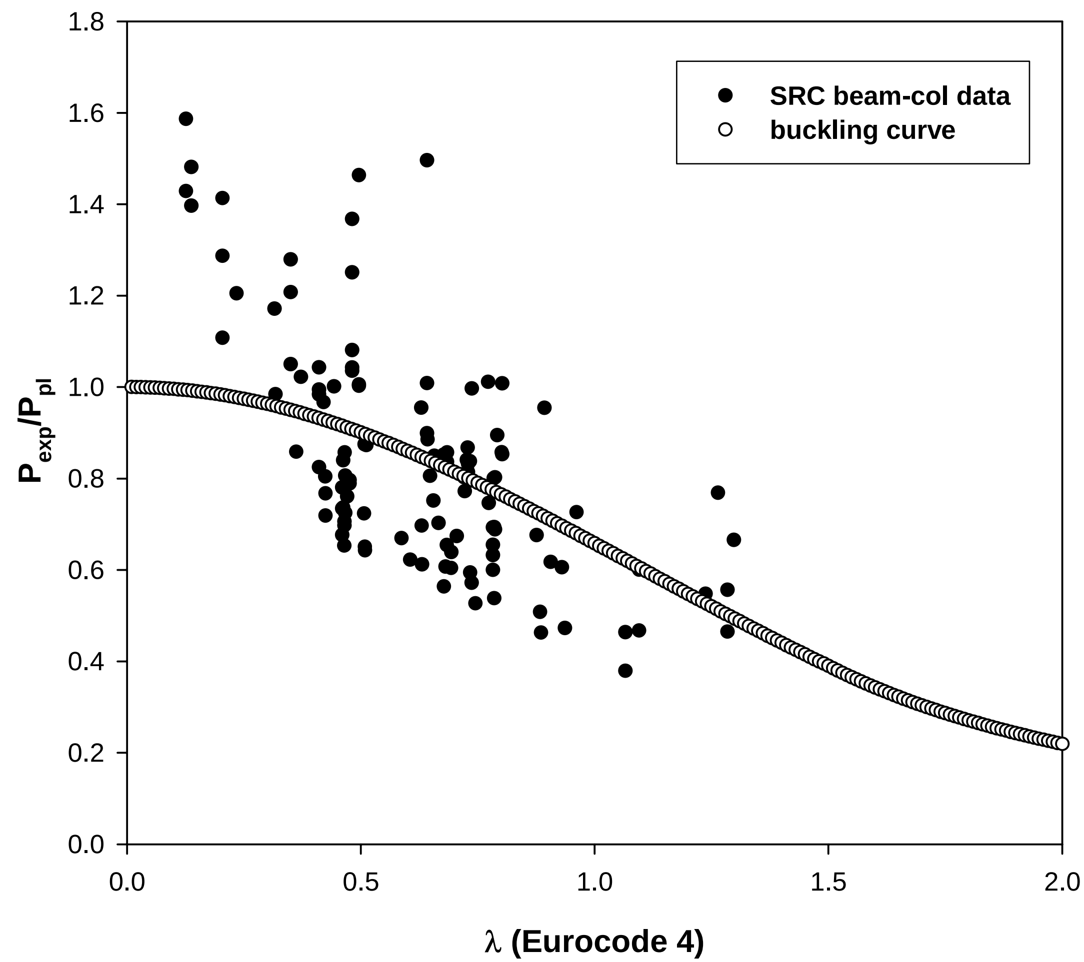


Figure 4-15 P_{exp}/P_{pl} with AISC buckling curve for SRC beam-columns by Eurocode 4

Table 4-2 Comparison of beam-column strengths, and α/λ

<table>
<tr><th colspan="2">Type</th><th>No. of Tests</th><th>Mean</th><th>Standard Deviation</th><th>COV</th><th>Mean Design Axial Load(AISC) 2005/1999 (COV)</th></tr>
<tr><td rowspan="5">Encased Beam-columns (SRC)</td><td>AISC 1999</td><td rowspan="5">117</td><td>1.41</td><td>0.32</td><td>0.23</td><td rowspan="2">0.82 (0.23)</td></tr>
<tr><td>AISC 2005</td><td>1.03</td><td>0.25</td><td>0.24</td></tr>
<tr><td>Eurocode 4</td><td>1.21</td><td>0.22</td><td>0.18</td><td rowspan="3"></td></tr>
<tr><td>α/λ (AISC 2005/ AISC 1999)</td><td>1.47</td><td>0.26</td><td>0.24</td></tr>
<tr><td>α/λ (AISC 2005/ Eurocode 4)</td><td>1.38</td><td>0.21</td><td>0.15</td></tr>
</table>

As shown in Table 4-2, the mean of the unfactored prediction by the AISC 1999 method was 1.41, standard deviation was 0.32, and coefficient of variation was 0.23. The mean value ranged from 0.88 to 2.45. When a resistance factor of 0.85 for compression and of 0.9 for bending were considered, the mean ratio was changed to 1.6, standard deviation was changed to 0.37 and coefficient of variation was changed to 0.23. By the AISC 2005 method, the mean ratio was 1.03, the standard deviation was 0.25, and the coefficient of variation was 0.24. The maximum was and minimum ratios were 1.98 and 0.62, respectively. When a resistance factor of 0.75 for compression and of 0.9 for bending were considered, the mean changed to 1.29 with a standard deviation of 0.34 and a coefficient of variation of 0.27. The mean of 1.41 by the AISC 1999 method was substantially larger than that of 1.03 by the AISC 2005 specification as shown in Table 4-2. This shows that predicted value by the AISC 2005 method is less conservative. The

AISC 2005 method performs very well when compared to the AISC 1999 because of its use of a polygonal path in predicting the interaction curve. One of the reasons why ratios by the AISC 1999 method have high values is that a majority of the specimens are stocky and have columns with small cross-sections. Comparison to the AISC 2005 method gave a better match to the experimental data.

The Eurocode also gave good predictions. The mean by Eurocode was 1.21 with a standard deviation of 0.22 and a coefficient of variation of 0.18. When a partial safety factor of 1.1 for the structural steel, of 1.5 for the concrete, of 1.15 for the reinforcing steel are used, the mean was 1.56 with a standard deviation of 0.29 and a coefficient of variance of 0.19.

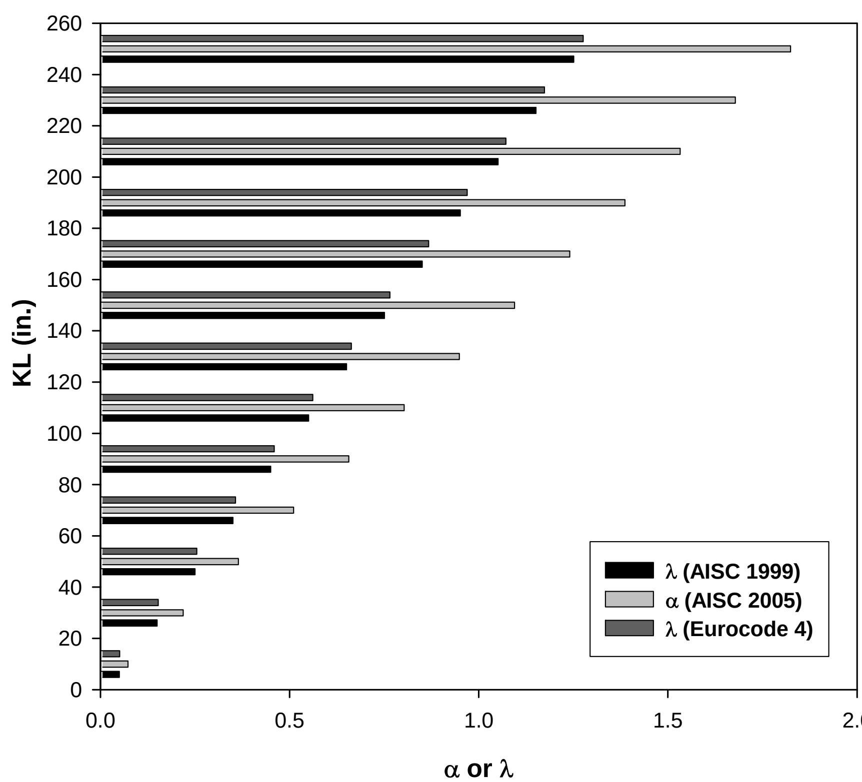


Figure 4-16 Comparison of slenderness ratio for a typical column in the SRC beam-column database (Janss and Anslijn, 1974)

From the standpoint of slenderness, the ratio of α/λ was 1.48 for the AISC 2005/AISC 1999 and 1.39 for the AISC 2005/ Eurocode 4. Figure 4-16 shows a comparison of slenderness ratios for 30th column in the database of SRC beam-columns given an effective length (Janss and Anslijn, 1974). The ratio of α/λ increases with column slenderness for the AISC methods, as shown in Figure 4-16. If the AISC 2005 and Eurocode 4 are compared, the AISC 2005 gives slightly larger values for λ for SRC beam-columns.

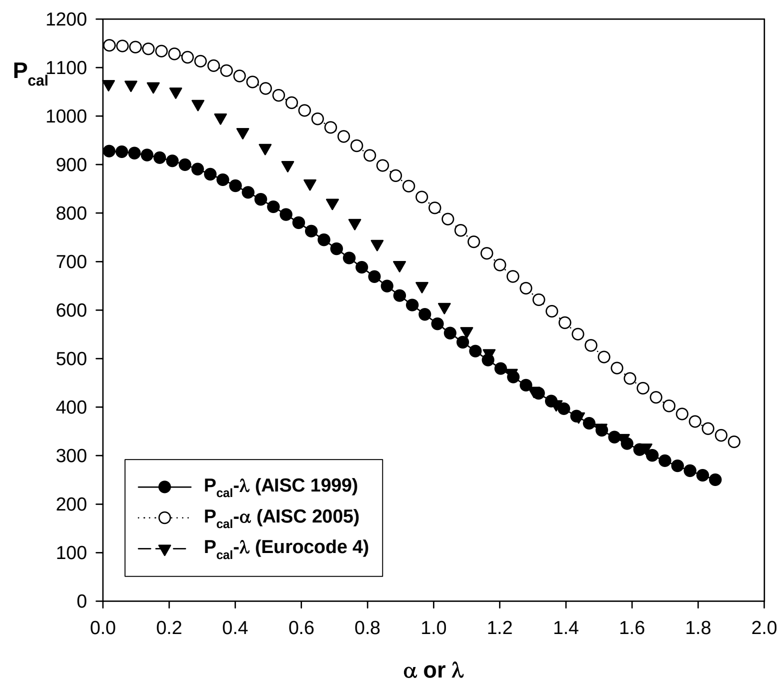


Figure 4-17 Axial capacity vs. slenderness for a typical beam-column in the SRC beam-column database (Roik, Mangerig, and Schwalbenhofer, 1990)

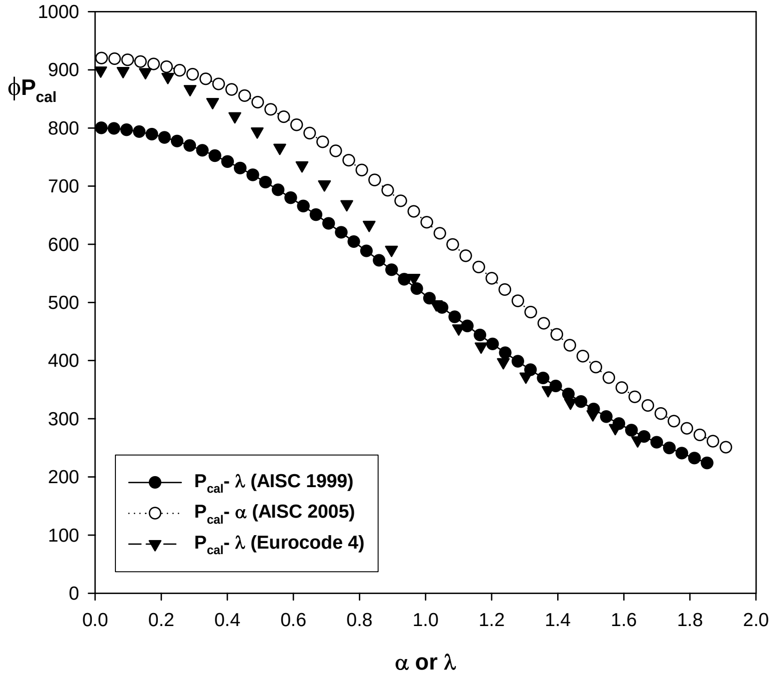


Figure 4-18 Axial capacity vs. slenderness for a typical beam-column in the SRC beam-column database (Roik, Mangerig, and Schwalbenhofer, 1990)

For a typical beam-column, Figure 4-17 indicates that P_{cal} by the AISC 2005 is larger than that by the AISC 1999, and that the difference increases as the slenderness increases (39th column, Roik, Mangerig, and Schwalbenhofer, 1990). The P_{cal} value by the AISC 2005 method is also larger than that by the AISC the 1999 if the design capacity (factored resistance) is compared. As shown in Table 4-2, the ratio of the design value between the AISC 1999 and 2005 is 0.82. However, the P_{cal} value by the Eurocode 4 is close to that by the AISC 2005 at low slenderness ratios and close to the P_{cal} value by the AISC 1999 at high slenderness ratio in SRC beam-columns.

4.1.3 CCFT Columns

A total of 312 axially loaded concrete-filled tube columns were collected in the database. From those, there were 67 CFT columns eliminated because they failed the local buckling limits for either the AISC specification or Eurocode 4. Two tests were not used because the ultimate axial load was not determined in the test (Chapman and Neogi, 1966), and three columns were removed because they had premature failures (Chapman and Neogi, 1966 ; Knolwes and Park, 1969). An additional four CFT columns were not used because the columns were very stocky and short (Chapman and Neogi, 1966). Six columns were eliminated because of the very small cross-section used (Salani and Sims, 1964). Twenty CFT columns were not used because they did not approach their ultimate axial capacity (Kilpatrick and. Rangan, 1997; Roeder and Cameron, 1999; Woo and Kim, [illegible]2). Finally, 210 CFT columns were used for the analyses and included in the reduced database. The range of material strengths and reinforcement ratios are shown in Figures 4-19 through 4-21. The steel yield stress ranged from 32.1 ksi to 121 ksi, while the compressive strength ranged from 2.6 ksi to 16.5 ksi. The range of yield stress and concrete compressive strength is fairly broad. The structural steel ratio varied between 5.5 % and 27 %. Figure 4.22 shows a 3D plot of the yield stress, concrete compressive strength and reinforcement ratio for beam-columns. This plot emphasizes the large gaps in the database around 9 ksi and 13 ksi in compressive strength of the concrete, more than 70 ksi of yield stress and over 0.25 of structural steel ratio. The diameter-to- thickness (D/t) ratio for CCFT columns ranged from 13.8 to 71.4, as shown in Figure 4-23. Figure 4-24, 4-25 and 4-26 show a scatter plot of the data by AISC 1999, AISC 2005 and Eurocode 4 versus the slenderness parameter.

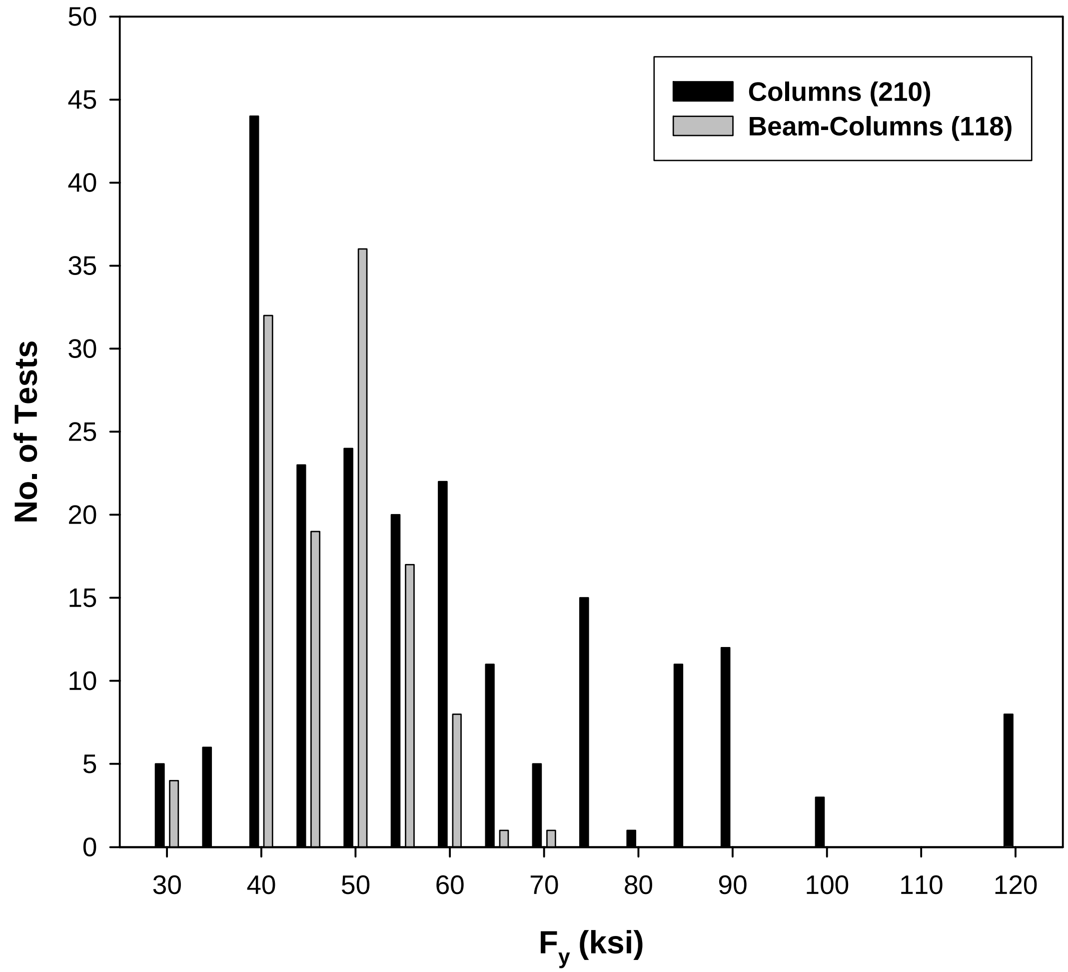


Figure 4-19 Frequency distribution of F_y for the reduced CCFT database

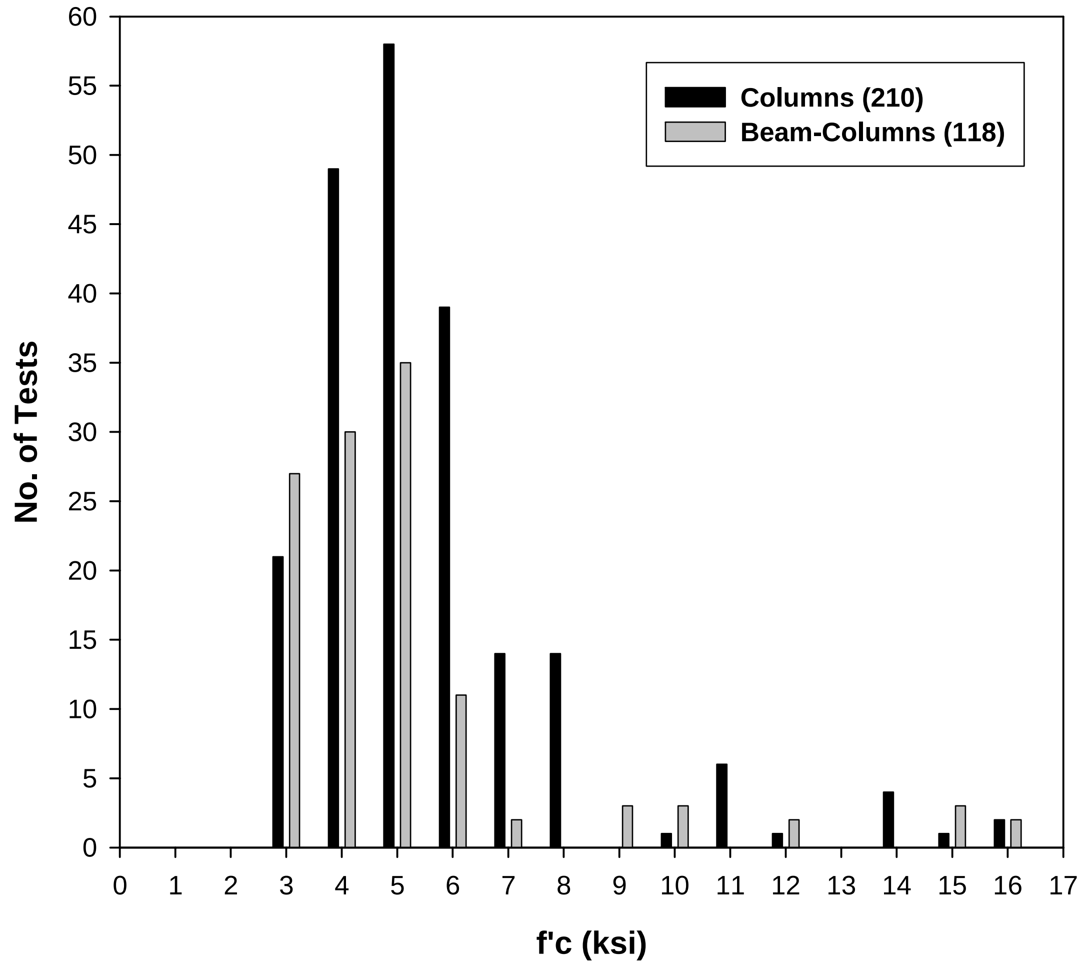


Figure 4-20 Frequency distribution of f'_c for the reduced CCFT database

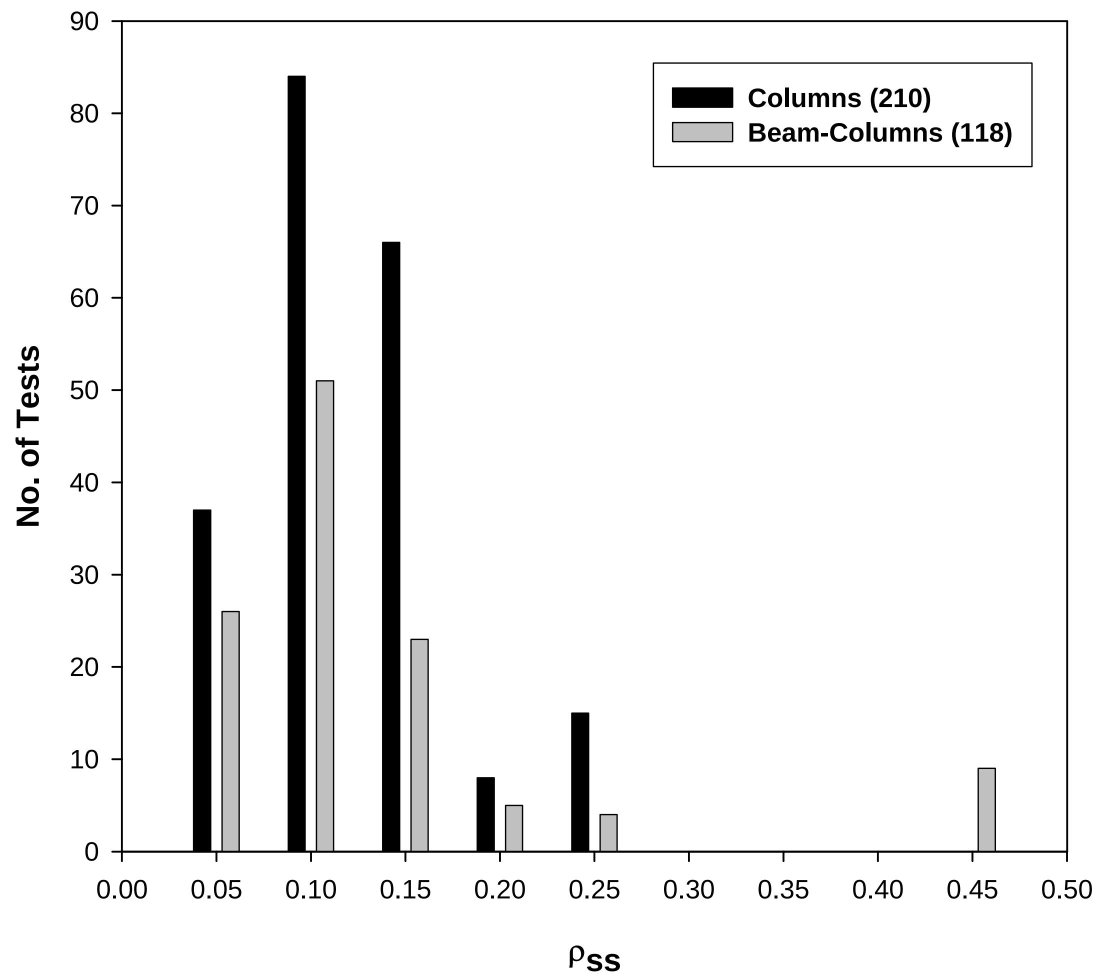


Figure 4-21 Frequency distribution of ρ_{ss} for the reduced CCFT database

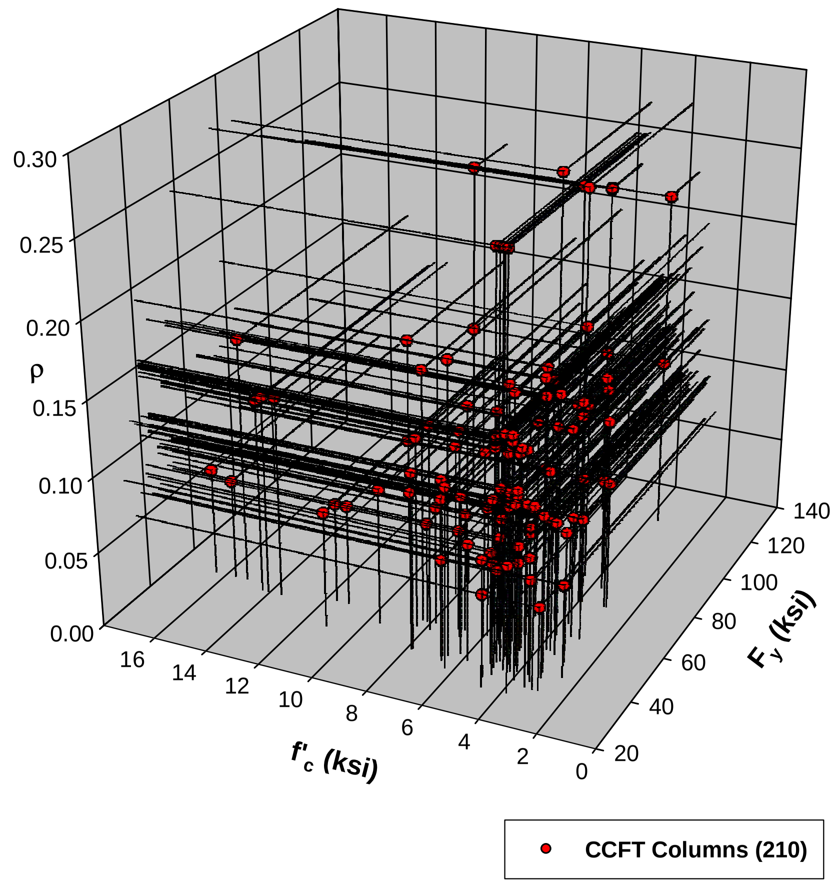


Figure 4-22 Frequency distribution of F_y, f'_c and ρ_{ss} for the reduced CCFT column database

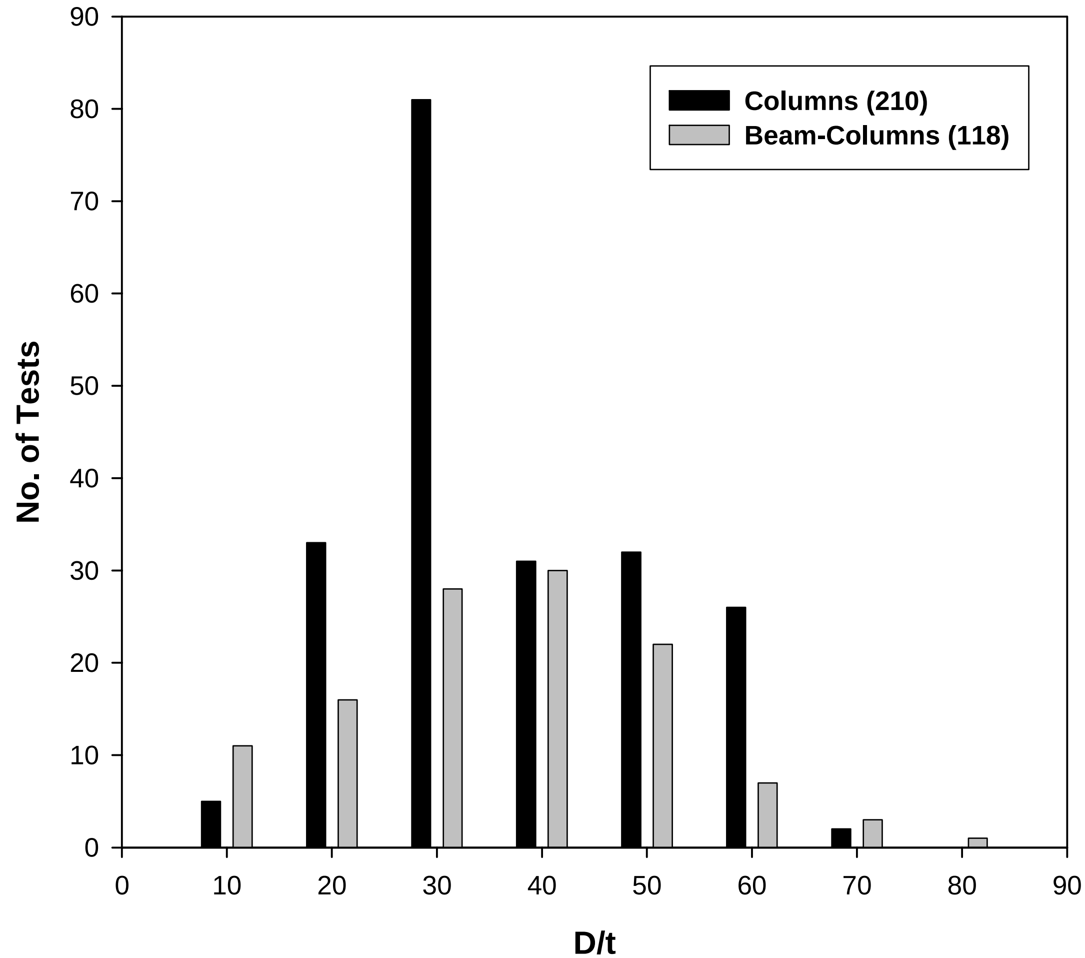


Figure 4-23 Frequency distribution of D/t for the reduced CCFT database

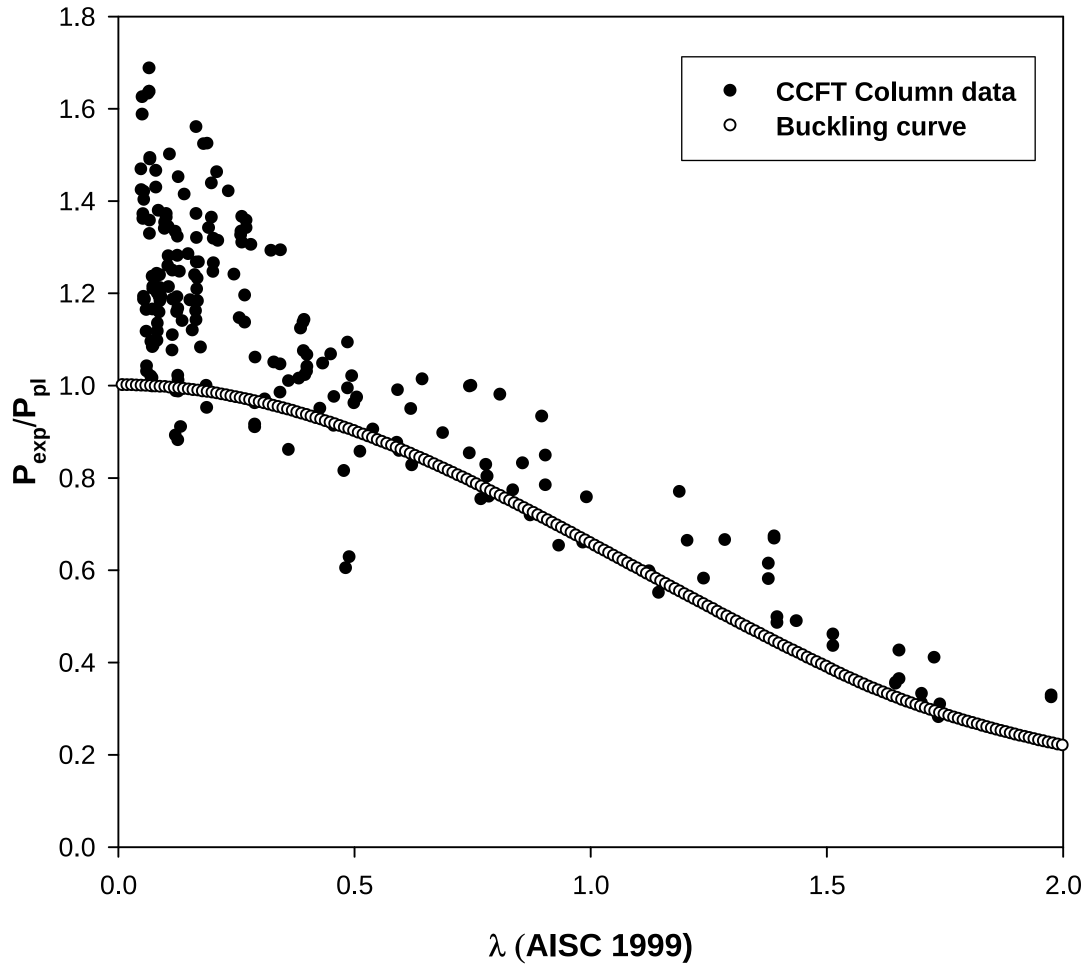


Figure 4-24 P_{exp}/P_{pl} with AISC buckling curve for CCFT columns by AISC 1999

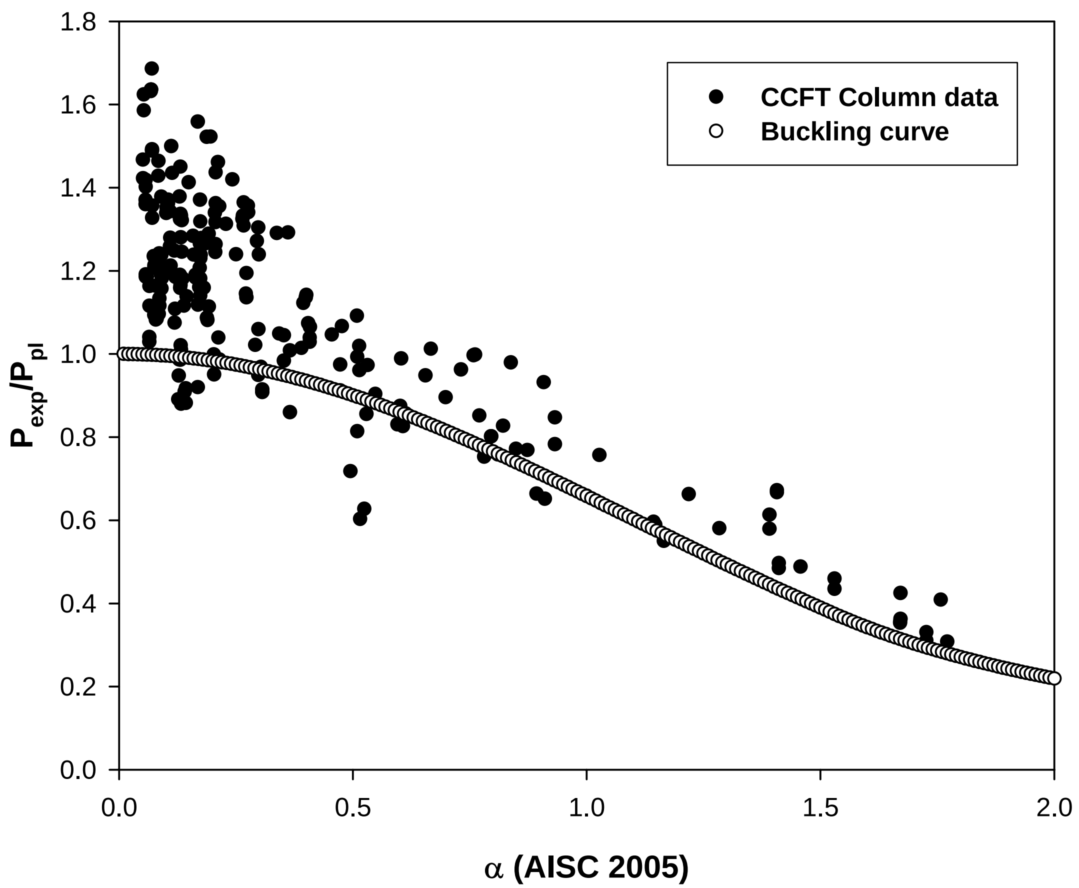


Figure 4-25 P_{exp}/P_{pl} with AISC buckling curve for CCFT columns by AISC 2005

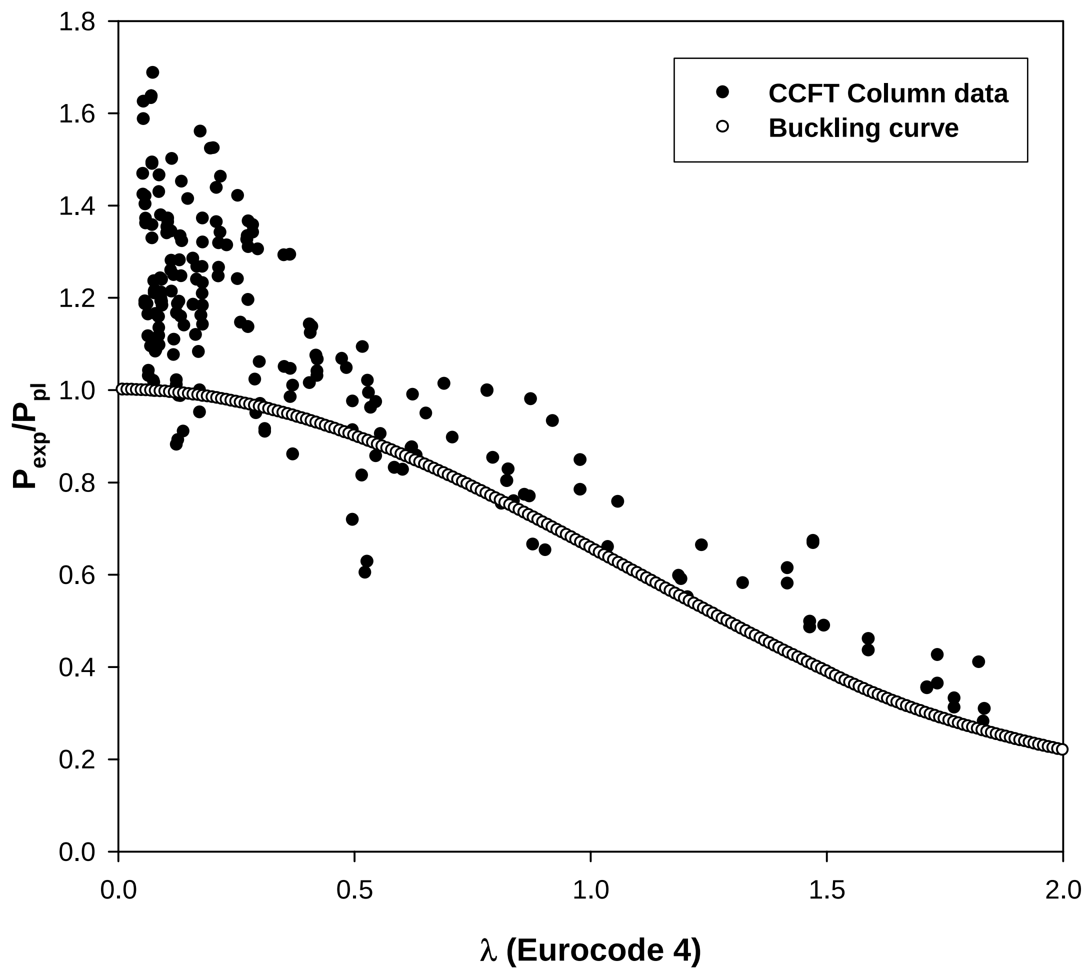


Figure 4-26 P_{exp}/P_{pl} with AISC buckling curve for CCFT columns by Eurocode 4

Table 4-3 Comparison of column strengths, and α/λ

Type		No. of Tests	Mean	Standard Deviation	COV	Mean Design Axial Load(AISC) 2005/1999 (COV)
Circular CF columns (CCFT)	AISC 1999	210	1.28	0.19	0.15	1.08 (0.04)
	AISC 2005		1.23	0.18	0.15	
	Eurocode 4		1.06	0.18	0.17	
	α/λ (AISC 2005/ AISC 1999)		1.03	0.06	0.06	
	α/λ (AISC 2005/ Eurocode 4)		0.99	0.05	0.05	

For unfactored capacities, the mean experimental to calculated axial capacity ratio by the AISC 1999 method was 1.28, the standard deviation was 0.19, and the coefficient of variation was 0.15. The ratio of experimental to predicted values ranged from 0.73 to 1.82. When a resistance factor of 0.85 was applied, the mean ratio increased to 1.51, the standard deviation to 0.22 and the coefficient of variation to 0.15. By the AISC 2005 method, the mean ratio was 1.23, the standard deviation was 0.18, and the coefficient of variation was 0.15. The ratio of experimental to predicted values ranged from 0.7 to 1.73. When a resistance factor of 0.75 was considered, the mean increased to 1.64 with a standard deviation of 0.25 and a coefficient of variation of 0.15. The mean of 1.28 by the AISC 1999 method was larger than that of 1.23 by the AISC 2005 specification. This means that the AISC 2005 method is less conservative and allows a higher axial capacity. One of the reasons for this difference is that this data set includes specimens with high strength concrete and high yield stress steel. In the AISC 1999 specification, the usable concrete strength had to be less than 8 ksi and that of the steel less than 60 ksi. However, the AISC 2005 specification liberalized those limits to 10 ksi for concrete and 75 ksi for steel. If specimens with high strength concrete and high yield stress steel are evaluated with the actual materials used, the values by AISC 1999 method have a higher mean compared with the AISC 2005 method.

The Eurocode predictions are very good for these cross-sections. The mean by the Eurocode 4 was 1.06 with a standard deviation of 0.18 and a coefficient of variation of 0.17. When a partial safety factor of 1.1 for the structural steel, of 1.5 for the concrete, of 1.15 for the reinforcing steel are used, the mean is 1.36 with a standard deviation of 0.2 and a coefficient of variation of 0.15. The Eurocode method predicts capacities very well even when high strength concrete and high yield stress is used.

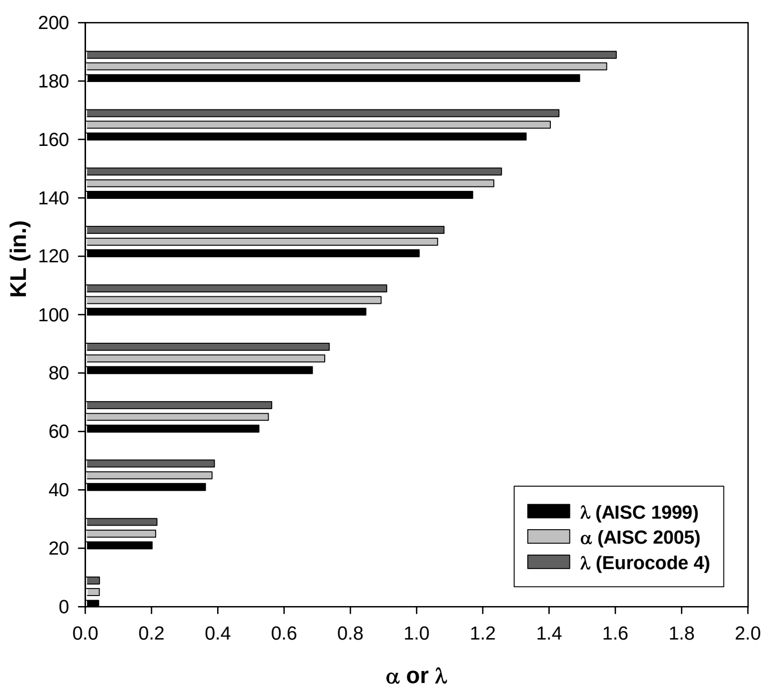


Figure 4-27 Comparison of slenderness ratio for a typical column in the CCFT database (Furlong, 1967)

Figure 4-27 shows a comparison of slenderness ratios for 66^{th} column in the circular CFT column database. When comparing slenderness factors, the ratio of α/λ was 1.03 (AISC 2005/AISC 1999) and 0.99 (AISC 2005/ Eurocode 4). Thus for circular concrete filled tube columns, the slenderness parameters λ and α are almost the same values for the AISC 1999, AISC 2005 and Eurocode 4.

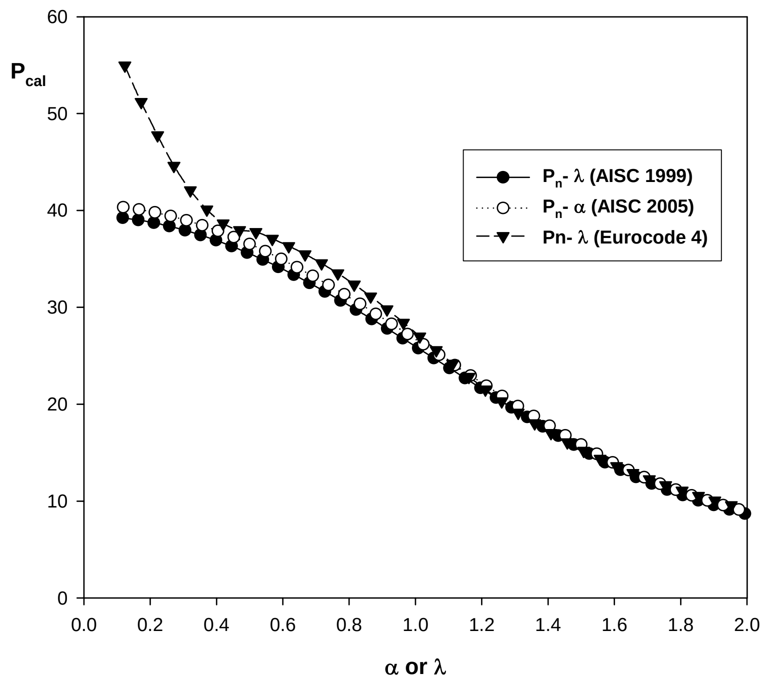


Figure 4-28 Axial capacity vs. slenderness for a typical [illegible]umn in the CCFT database (Salani and Sims, 1964)

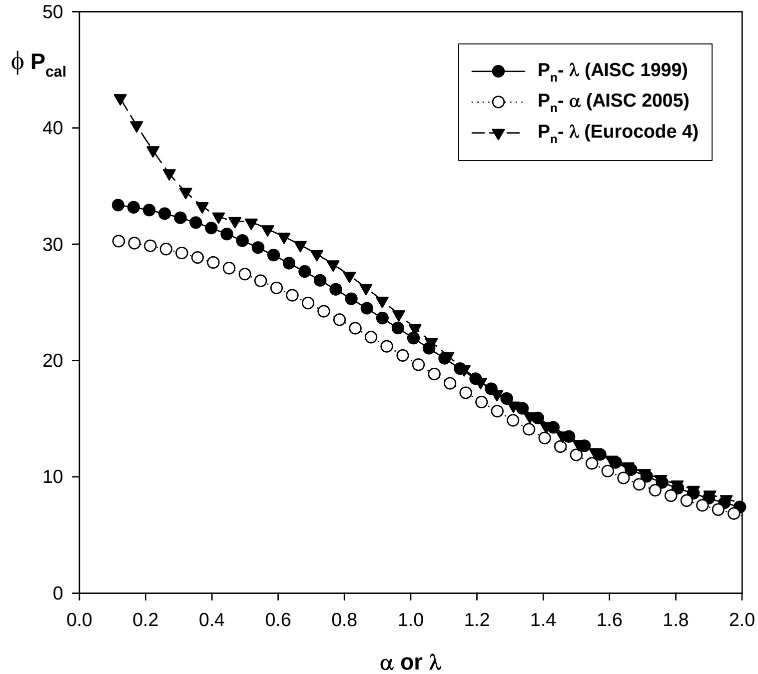


Figure 4-29 Axial capacity vs. slenderness for a typical column in the CCFT database (Salani and Sims, 1964)

As shown Figure 4-28, the P_{cal} value by the AISC 2005 is slightly larger than that by the AISC 1999 for the entire slenderness range if the comparisons are made without the resistance factor (88^{th} column, Salani and Sims, 1964). When comparing design values, which include the resistance factor, the mean value by the AISC 1999 method is larger than that by AISC 2005 method. The mean design load for the AISC 2005/1999 methods is 1.08 and the AISC 1999 curve lies higher than that of the AISC 2005 method in Figure 4-29. For the Eurocode 4, P_n is much larger when λ is less than 0.5, because the additional effect of confinement is explicitly considered.

4.1.3 CCFT Beam-Columns

There were a total of 198 concrete-filled tube beam-columns in the database. From those, 27 CCFT beam-columns were eliminated when the cross-sections failed to meet the local buckling check of either the AISC specification or the Eurocode (Furlong, 1967; Kvedaras and Tomaszewicz, 1994; O'Shea and Bridge, 2000). Twenty-seven beam-columns were removed because they had unequal end moments and double curvature. Fourteen CCFT beam-columns were not used because they did not approach their maximum capacity in the tests (Knowles and Park, 1969; Jung, Choi and Shin, 1994). Nine beam-columns were eliminated because the values of yield stress and concrete compressive strength could not be reliably established (Rangan and Joyce, 1992). One test was discarded because the concrete was not well consolidated in the specimen (Furlong, 1967). Two tests were not used because the section size was very small (Kilpatrick and Rangan, 1997). Finally, 118 CCFT beam-columns were used for analysis and included in the reduced database.

As shown in Figure 4-19, the yield stress ranged from 27.5 ksi to 70 ksi. The maximum compressive strength was 16.3 ksi and the minimum was 2.9 ksi (Figure 4-20). The structural steel ratio ranged from 5.1% to 46.6% as shown in Figure 4-21. Figure 4-30 shows a 3D plot of the yield stress, concrete compressive strength and reinforcement ratio for the CCFT beam-columns. This plot emphasizes the large gaps in the database around 11 ksi and 14 ksi in compressive strength of the concrete, more than 70 ksi of yield stress and around 0.25 of structural steel ratio. The D/t ratio for the CCFT beam-columns, as shown in Figure 4-23, varied between 7.4 and 78.1. The ratio for e/D ranged from 0.003 to 1.41 as shown in Fig. 4-31. Figures 4-32, 4-33 and 4-34 show scatter plots of the data by AISC 1999, AISC 2005 and Eurocode 4 versus the slenderness parameter, respectively.

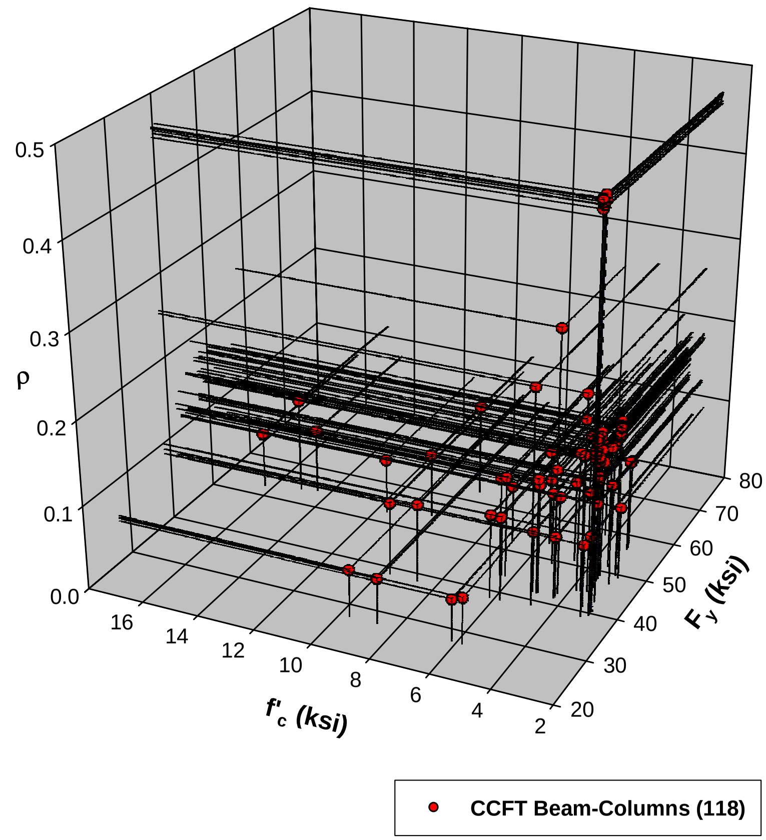


Figure 4-30 Frequency distribution of F_y, f'_c and ρ_{ss} for the reduced CCFT database

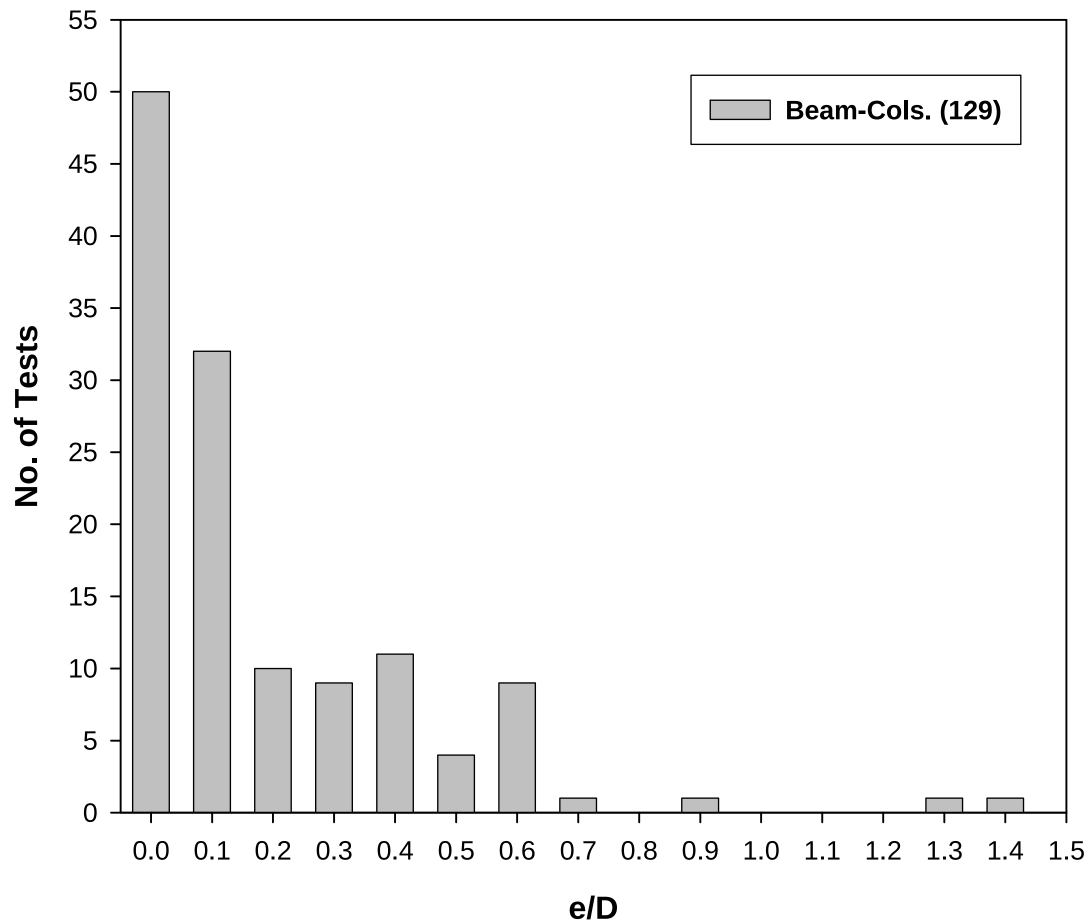


Figure 4-31 Frequency distribution of e/D for the reduced CCFT database

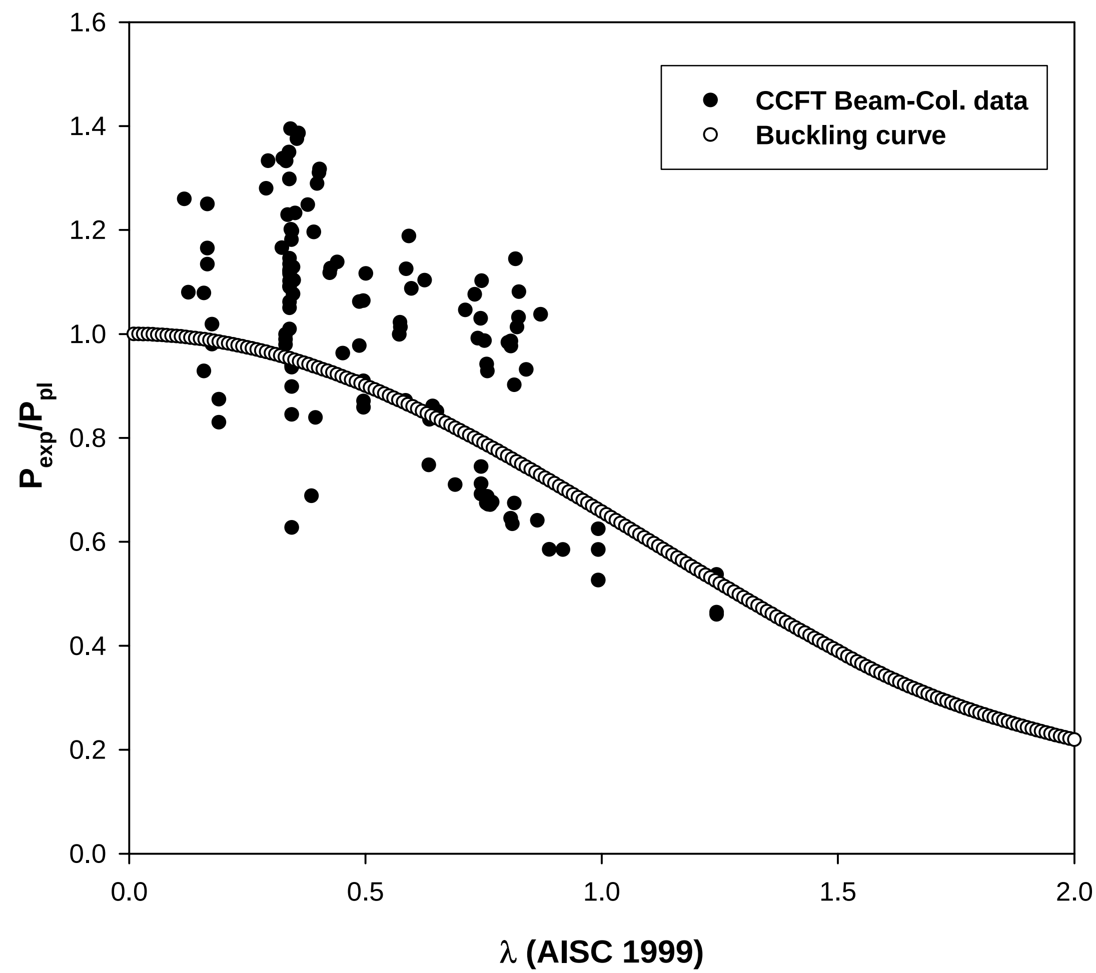


Figure 4-32 P_{exp}/P_{pl} with AISC buckling curve for CCFT beam-columns by AISC 1999

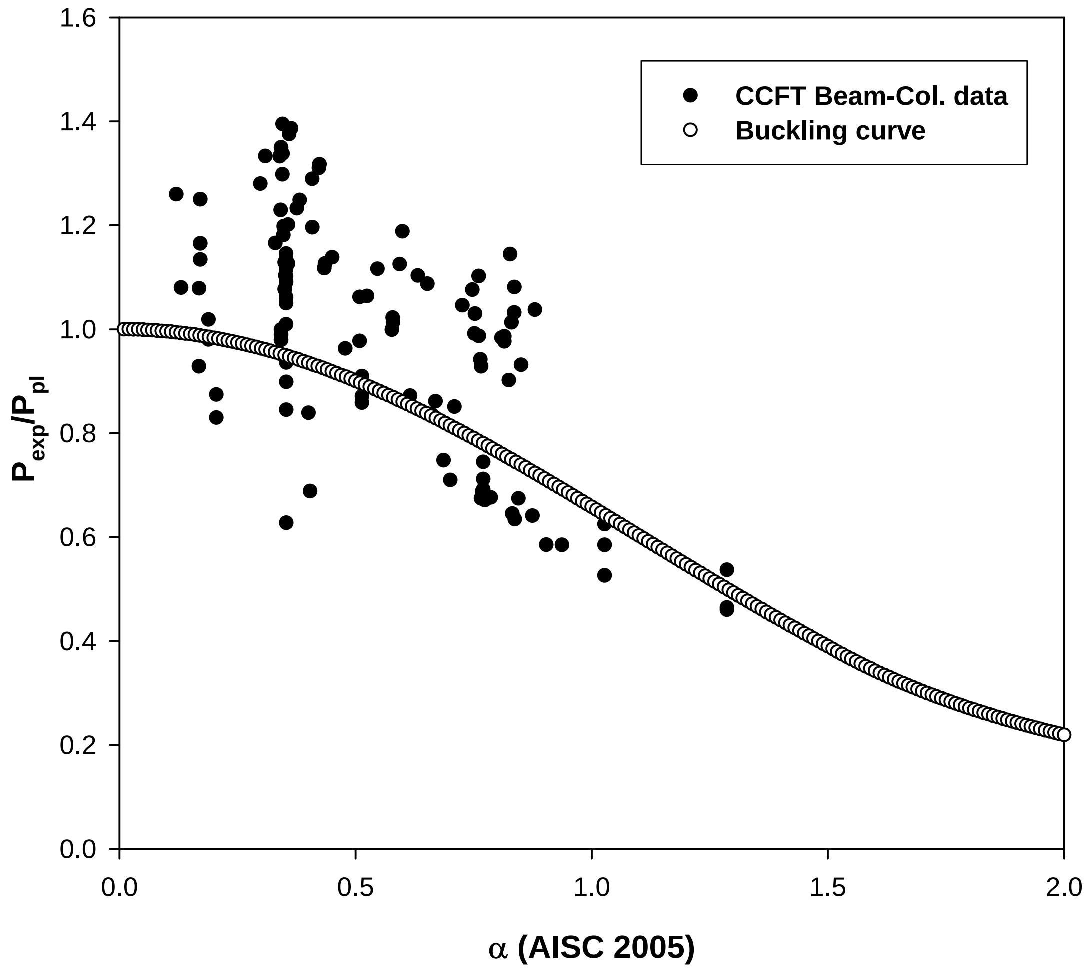


Figure 4-33 P_{exp}/P_{pl} with AISC buckling curve for CCFT beam-columns by AISC 2005

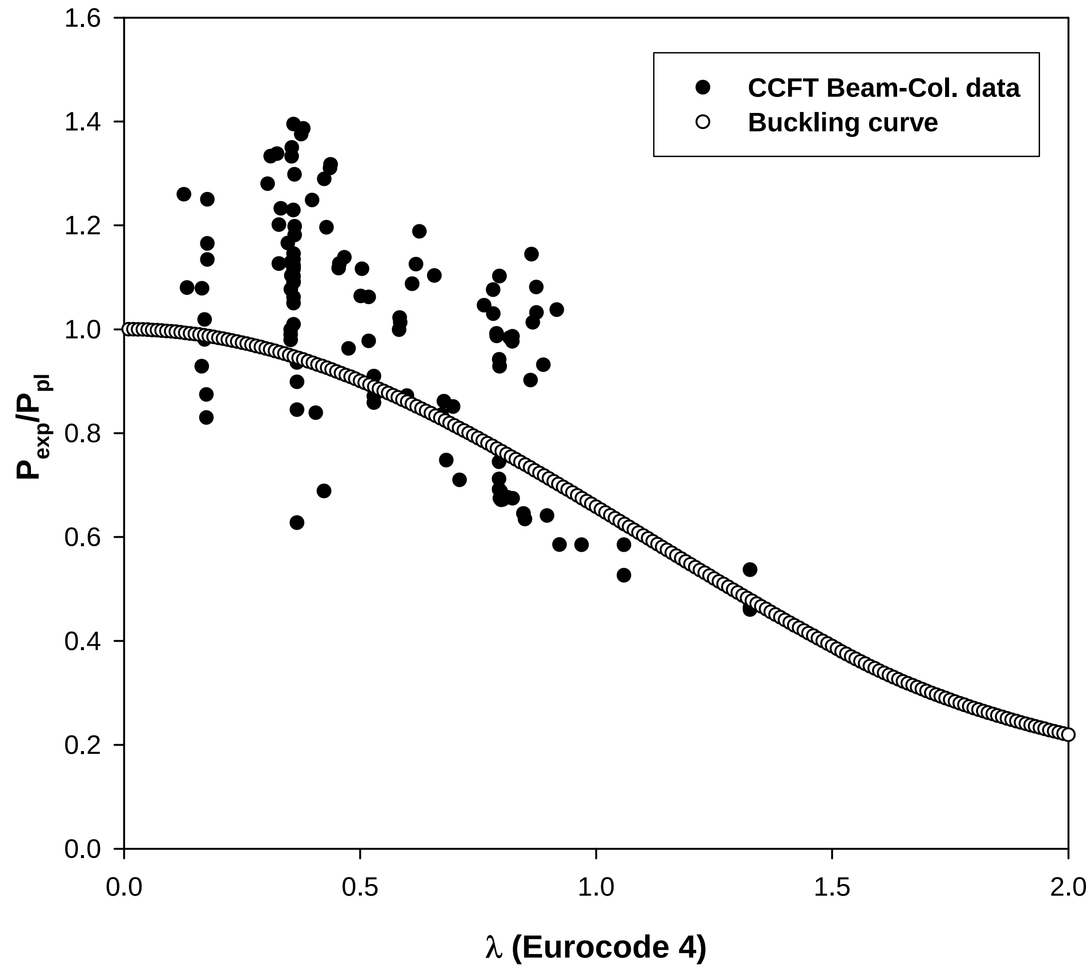


Figure 4-34 P_{exp}/P_{pl} with AISC buckling curve for CCFT beam-columns by Eurocode 4

Table 4-4 Comparison of Beam-column strengths, and α/λ

<table>
<tr><th colspan="2">Type</th><th>No. of Tests</th><th>Mean</th><th>Standard Deviation</th><th>COV</th><th>Mean Design Axial Load(AISC) 2005/1999 (COV)</th></tr>
<tr><td rowspan="5">Circular CF Beam-columns (CCFT)</td><td>AISC 1999</td><td rowspan="5">118</td><td>1.49</td><td>0.33</td><td>0.22</td><td rowspan="2">0.88 (0.21)</td></tr>
<tr><td>AISC 2005</td><td>1.14</td><td>0.22</td><td>0.19</td></tr>
<tr><td>Eurocode 4</td><td>1.25</td><td>0.19</td><td>0.15</td><td rowspan="3"></td></tr>
<tr><td>α/λ (AISC 2005/ AISC 1999)</td><td>1.03</td><td>0.02</td><td>0.02</td></tr>
<tr><td>α/λ (AISC 2005/ Eurocode 4)</td><td>0.99</td><td>0.04</td><td>0.04</td></tr>
</table>

For unfactored predictions, the mean experimental to calculated axial capacity ratio by the AISC 1999 method was 1.49, the standard deviation was 0.33, and the coefficient of variation was 0.22. The maximum ratio was 2.43 and minimum ratio was 0.83. When a resistance factor of 0.85 for compression and of 0.9 for bending were considered, the mean ratio was changed to 1.7, the standard deviation to 0.37 and coefficient of variation to 0.22. By the AISC 2005 method, the mean ratio was 1.14, the standard deviation was 0.22, and the coefficient of variation was 0.19. The ratio of experimental to predicted values ranged from 0.72 to 1.55. When a resistance factor of 0.75 for compression and of 0.9 for bending were considered, the mean increased to 1.46, with a standard deviation of 0.32 and a coefficient of variation of 0.22. The AISC 1999 method gives comparatively lower values and can be very conservative, as shown in Table 4-4. Also, if high strength concrete (>8 ksi) specimens are compared, the ratio by AISC 1999 is 1.91 and the ratio by AISC 2005 is 1.09. This shows that the AISC 1999 method does not work well for high strength concrete.

The mean by the Eurocode 4 was 1.25, with a standard deviation of 0.19 and a coefficient of variation of 0.15. When partial safety factors of 1.1 for the structural steel, of 1.5 for the concrete, of 1.15 for the reinforcing steel are used, the mean is 1.5 with a standard deviation of 0.23 and a coefficient of variance of 0.15. For high strength concrete (>8 ksi) specimens, the ratio by the Eurocode is 1.28. This shows that the Eurocode 4 predictions also work very well for high strength concrete.

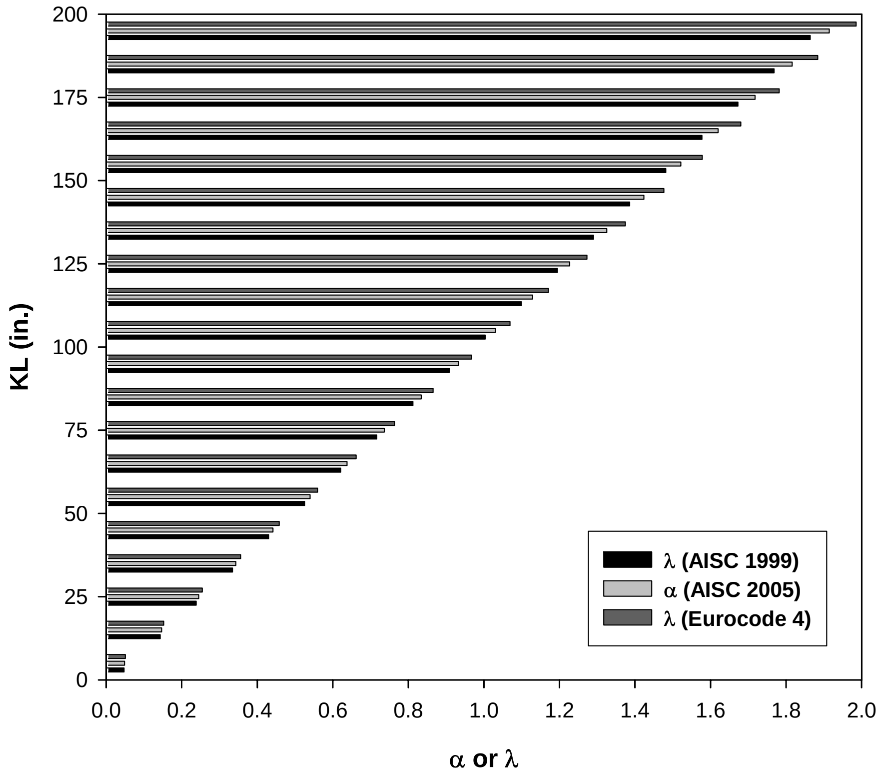


Figure 4-35 Comparison of slenderness ratio for a typical beam-column in the CCFT beam-column database (Furlong, 1967)

Figure 4-35 shows a comparison of slenderness ratios for 2nd beam-column in the database of CFT beam-columns given an effective length. When comparing slenderness, the ratio of α/λ was 1.04 for the AISC 2005/AISC 1999 and 1.0 for the AISC 2005/ Eurocode 4 comparisons. For circular concrete filled tube beam-columns, the slenderness parameters λ and α are almost the same value for the AISC 1999, AISC 2005 and Eurocode 4.

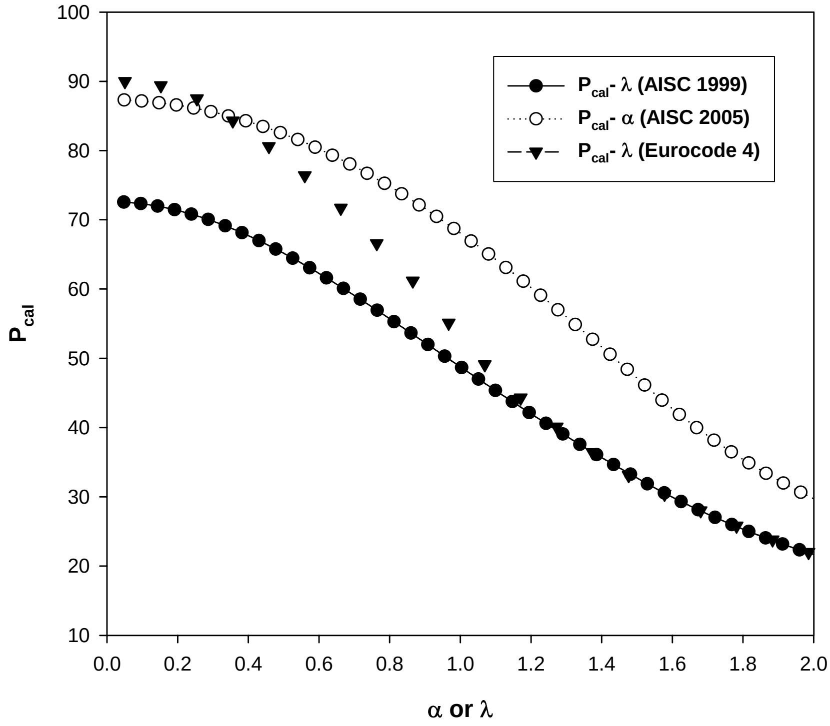


Figure 4-36 Axial capacity vs. slenderness for a typical beam-column in the CCFT beam-column database (Furlong, 1967)

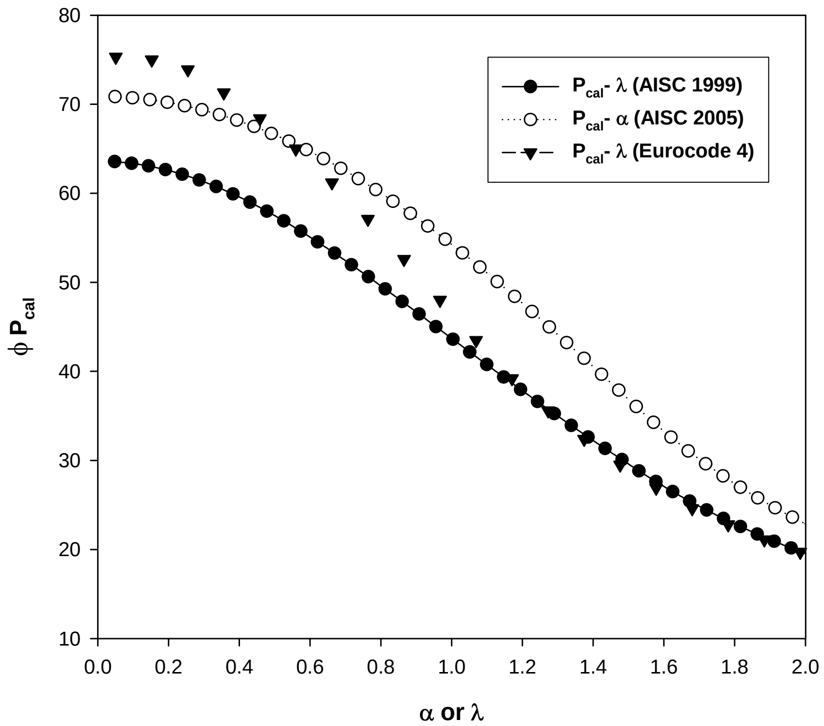


Figure 4-37 Axial capacity vs. slenderness for a typical beam-[illegible]umn in the CCFT beam-column database (Furlong, 1967)

As shown in Figures 4-[illegible]nd 4-37, the P_{cal} value by the AISC 2005 method is larger than that by the AISC [illegible]9 at all slendernesses, both with and without resistance factors (2nd column, Furlong, 1967). For design, the mean value by the AISC 2005 method is larger than that by AISC 1999 method, with a ratio of AISC 1999 to 2005 of 0.88 and the AISC 2005 curve is higher than the corresponding AISC 1999 one in Figure 4-37. For the Eurocode 4, the P_{cal} is a large value when λ is less than 0.5, because of the additional effect of confinement, which the Eurocode considers explicitly.

4.1.5 RCFT Columns

There were a total of 222 rectangular concrete-filled tube columns in the database. From those, 103 RCFT columns were eliminated as they failed the local buckling check of both AISC specification and Eurocode 4. For four specimens, reinforcing bars were used inside the tube (Grauers, 1993); since this case was not explicitly considered in developing the design equations, these specimens were removed. Three columns were removed because they had a high yield stress and a very small section (Uy, 2000). Two RCFT columns were not used because columns were stocky and short (Kang, Lim, and Moon, 2002). Six columns were eliminated because of small cross-section (Lee, Park, Kim, 2002). One test was eliminated because of problems in test (Chapman and Neogi, 1966). Finally, 103 CFT columns were used for analysis and included in the reduced database.

As shown in Figure 4-38, the yield stress ranged from 36.9 ksi to 120.8 ksi. The compressive strength ranged from 2.6 ksi to 14.9 ksi. The structural steel ratio ranged from 7.1% to 26.6% as shown in Fig. 4-40. Figure 4.41 shows a 3D plot of the yield stress, concrete compressive strength and reinforcement ratio for beam-columns. This plot emphasizes the large gaps in the database around 12 ksi in compressive strength of the concrete, more than 70 ksi of yield stress and around 0.05 of structural steel ratio. The distribution of B/t ratios had maximum value of 55.7 and minimum value of 13.9 as shown in Fig. 4-42. Figures 4-43, 4-44 and 4-45 show scatter plots of the data by AISC 1999, AISC 2005 and Eurocode 4 versus the slenderness parameter, respectively.

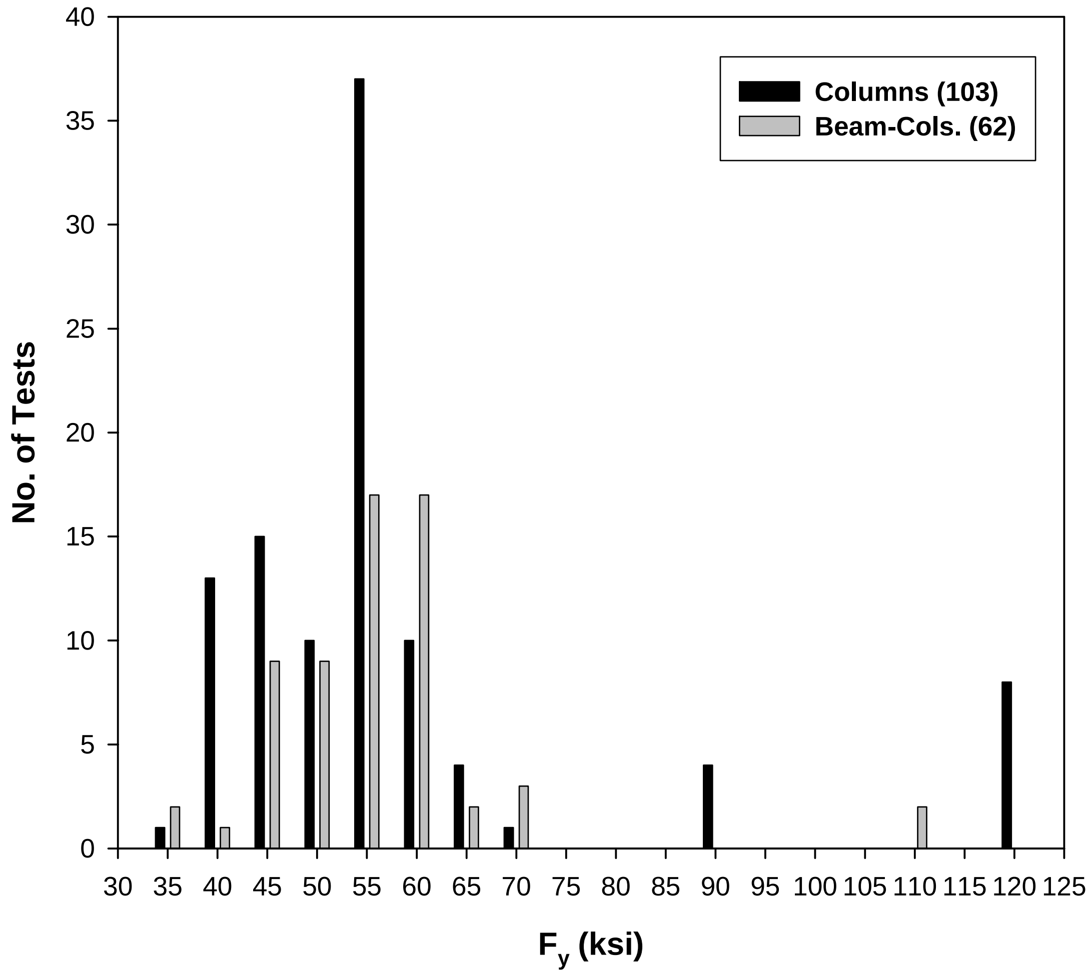


Figure 4-38 Frequency distribution of F_y for the reduced RCFT database

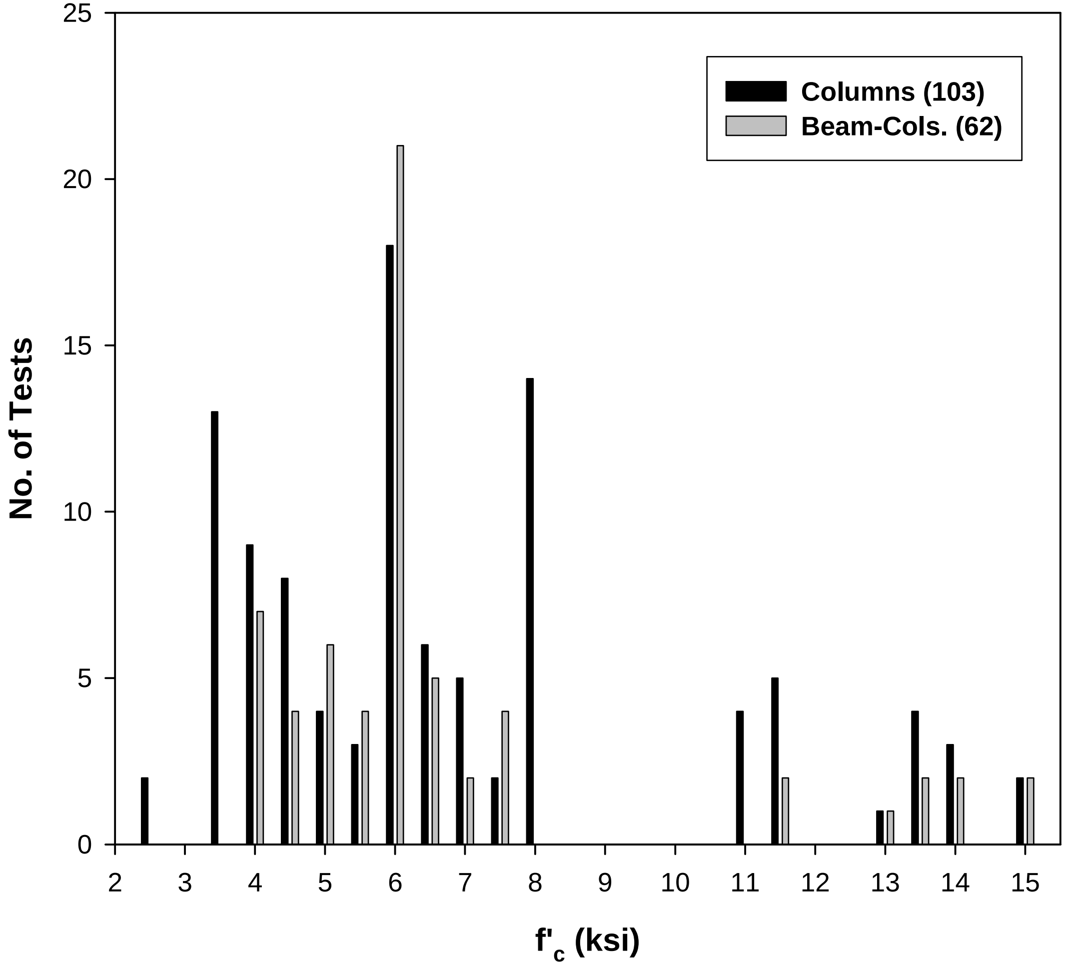


Figure 4-39 Frequency distribution of f'_c for the reduced RCFT database

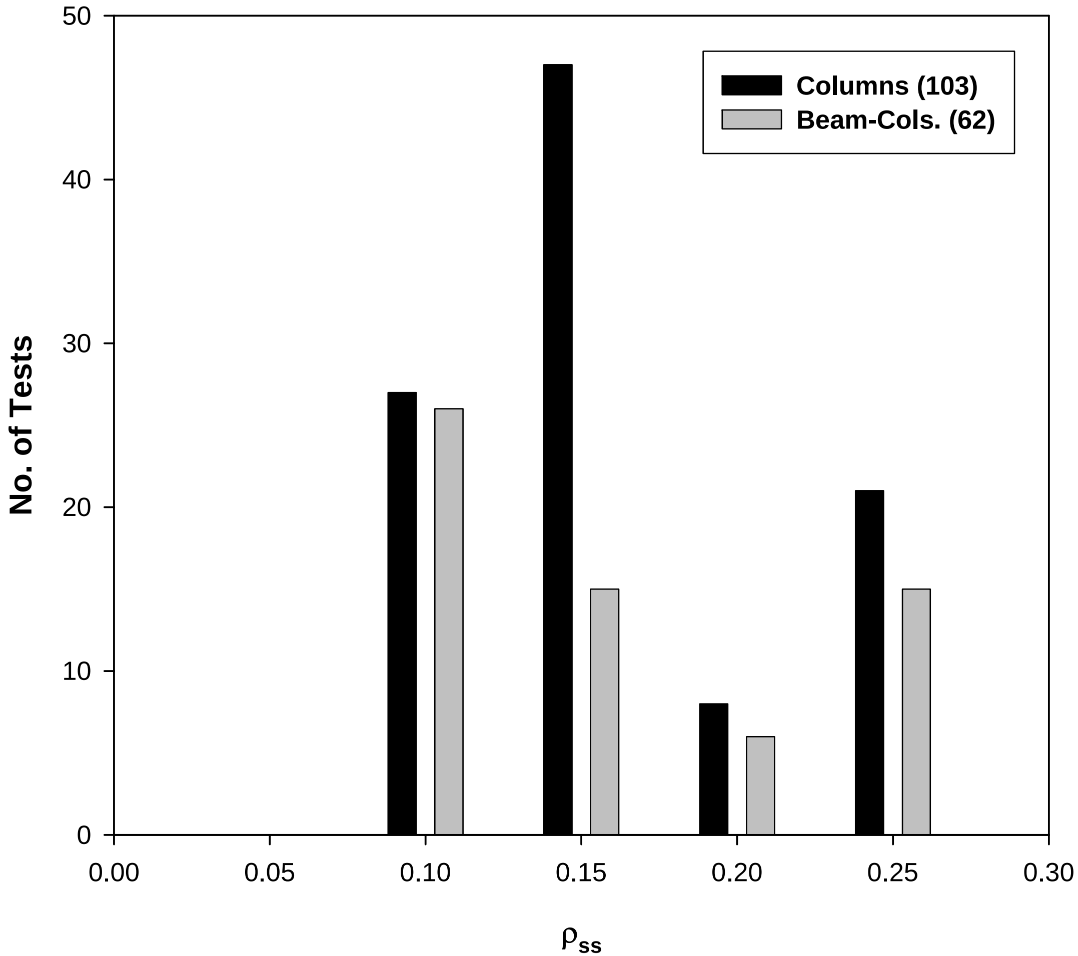


Figure 4-40 Frequency distribution of ρ_{ss} for the reduced RCFT database

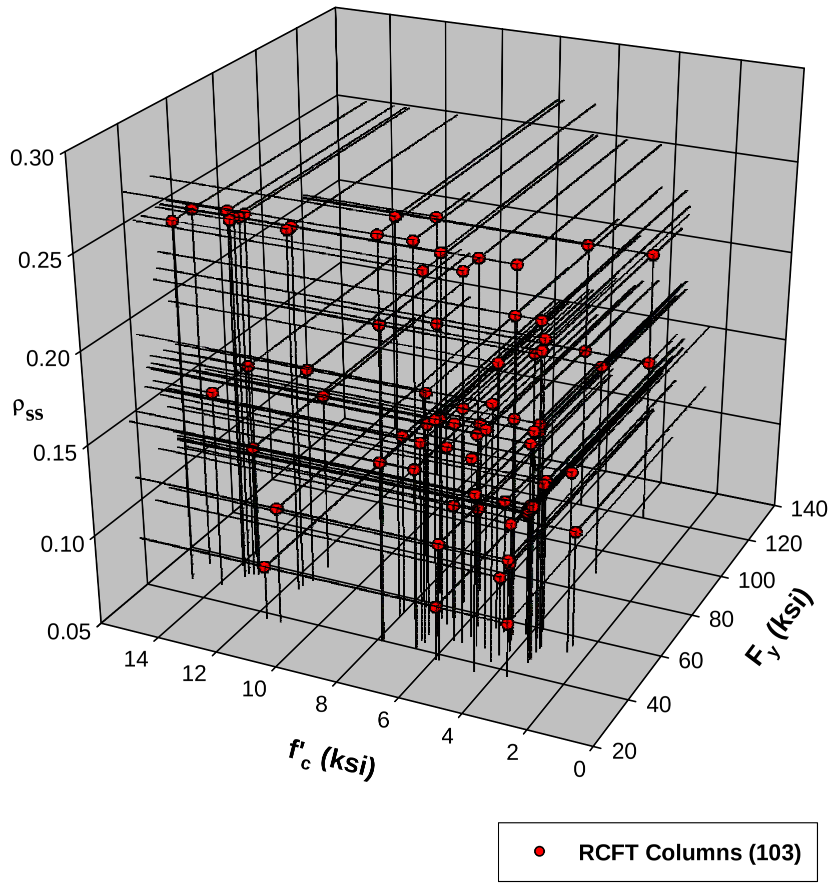


Figure 4-41 Frequency distribution of F_y, f'_c and ρ_{ss} for the reduced CCFT column database

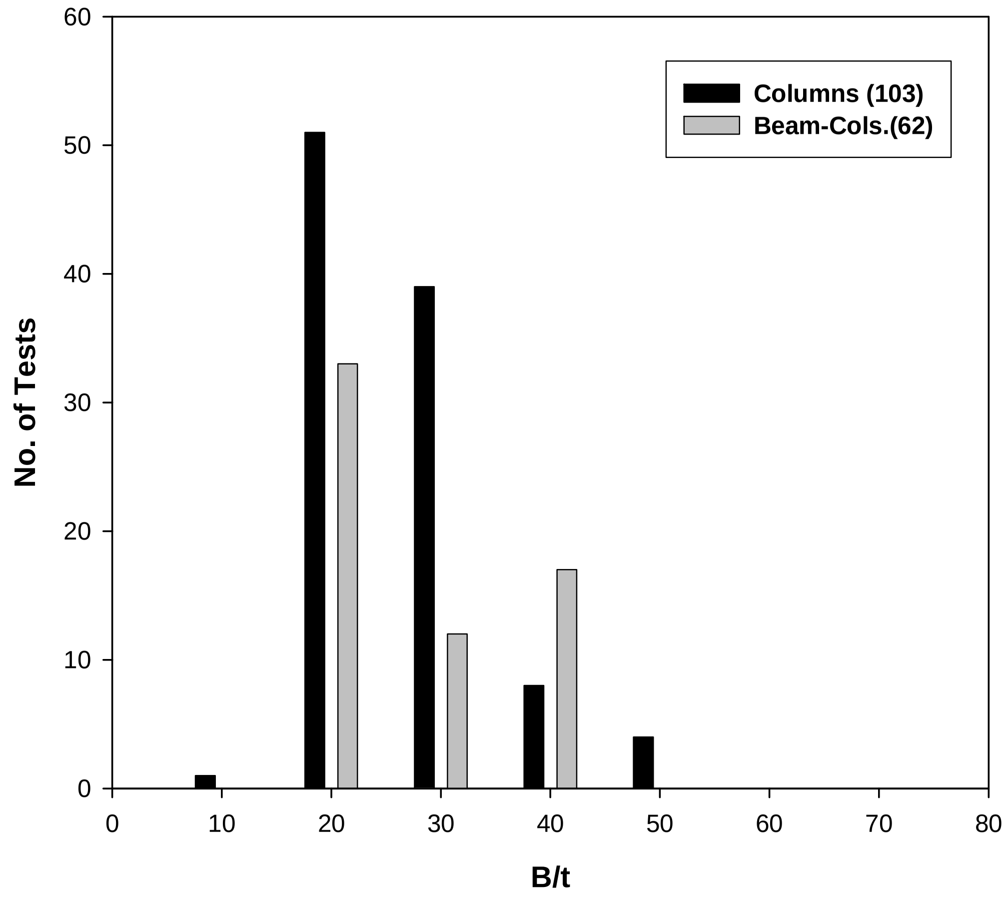


Figure 4-42 Frequency distribution of B/t for the rReduced RFT database

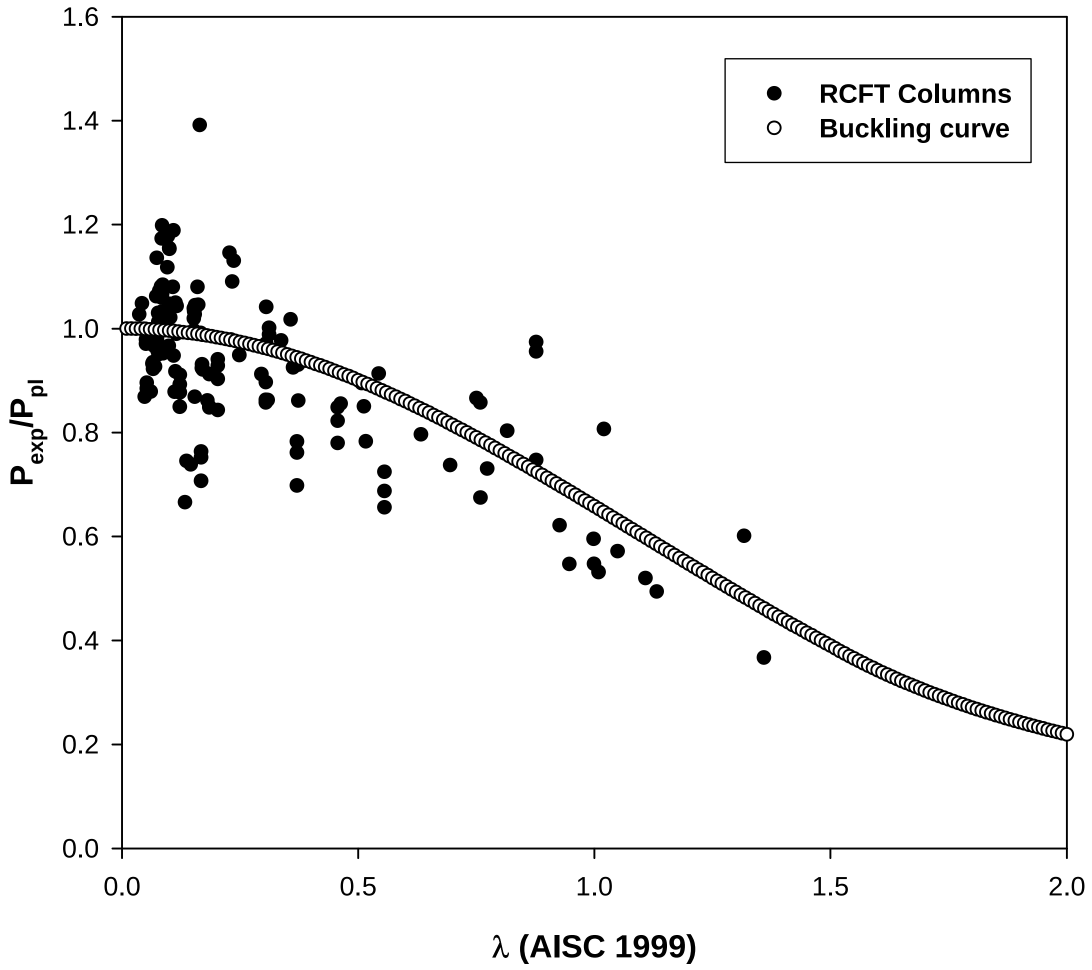


Figure 4-43 P_{exp}/P_{pl} with AISC buckling curve for RCFT columns by AISC 1999

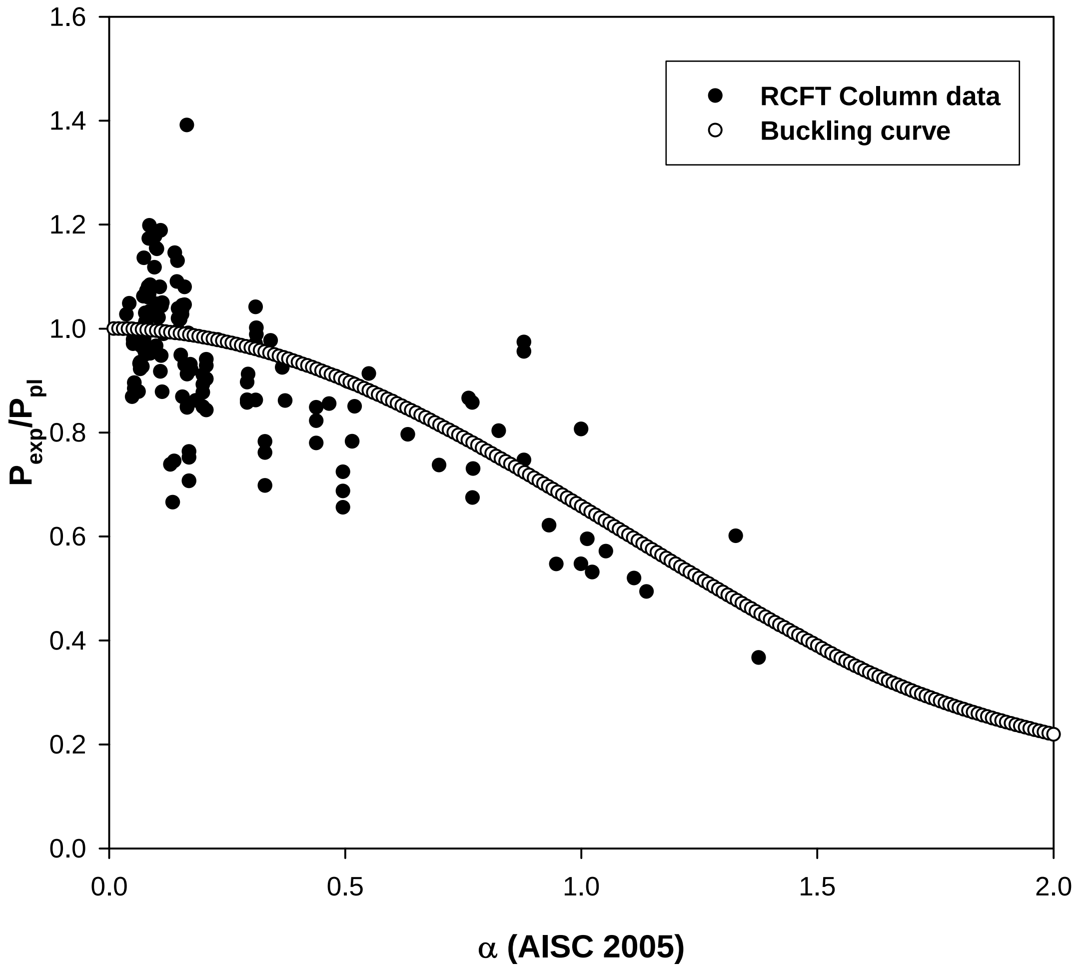


Figure 4-44 P_{exp}/P_{pl} with AISC buckling curve for RCFT columns by AISC 2005

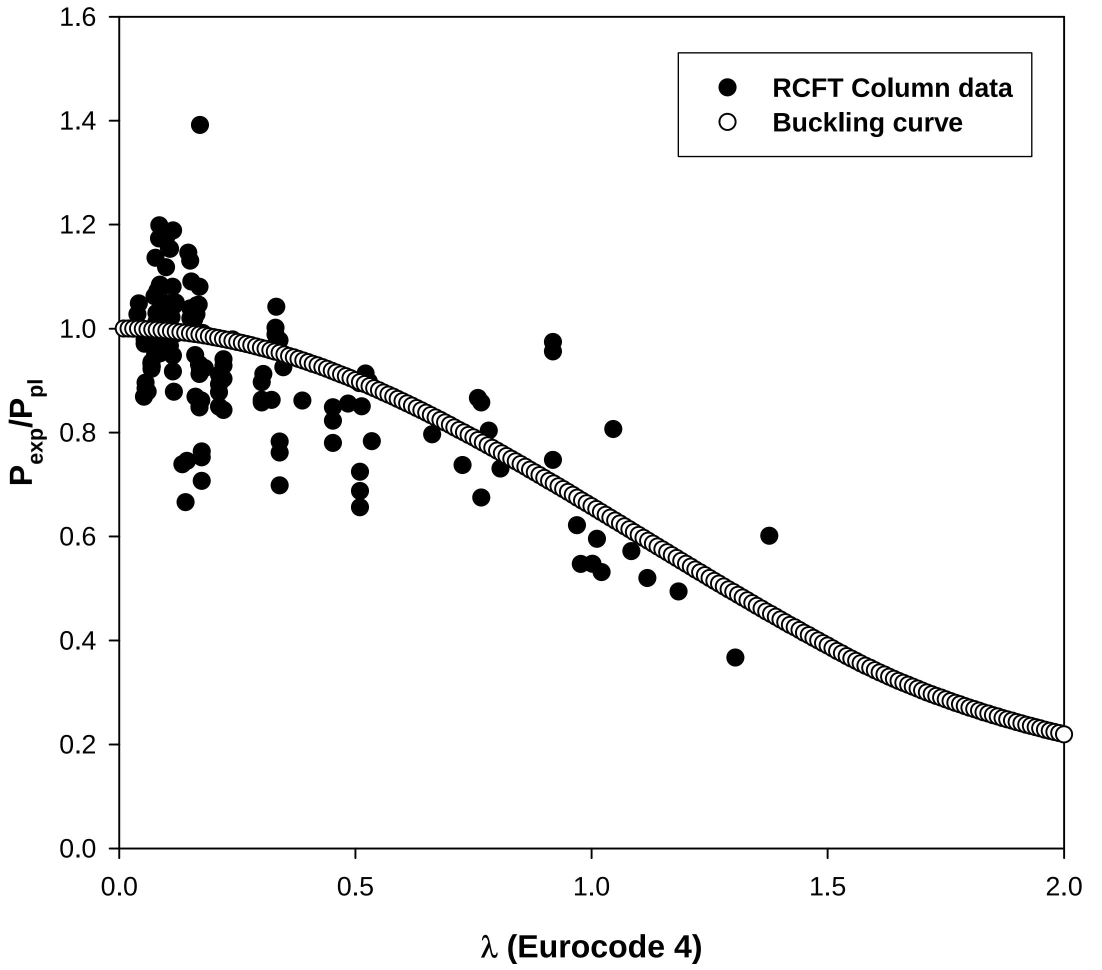


Figure 4-45 P_{exp}/P_{pl} with AISC buckling curve for RCFT columns by Eurocode 4

Table 4-5 Comparison of column strengths, and α/λ

Type		No. of Tests	Mean	Standard Deviation	COV 2005 (1999)	Mean Design Axial Load(AISC) 2005/1999 (COV)
Rectangular CF columns (RCFT)	AISC 1999	103	1.06	0.12	0.11	1.13 (0.01)
	AISC 2005		1.06	0.12	0.11	
	Eurocode 4		0.99	0.12	0.12	
	α/λ (AISC 2005/ AISC 1999)		0.99	0.14	0.14	
	α/λ (AISC 2005/ Eurocode 4)		0.97	0.13	0.13	

The mean experimental to calculated axial capacity ratio by the AISC 1999 method for unfactored capacity was 1.06, the standard deviation was 0.12, and the coefficient of variation was 0.11. The ratio of experimental to calculated values varied from 0.72 to 1.45. When a resistance factor of 0.85 was applied, the mean ratio increased to 1.24, the standard deviation to 0.14 and the coefficient of variation to 0.11. By the AISC 2005 method, the mean ratio was 1.06, the standard deviation was 0.12, and the coefficient of variation was 0.12. The maximum ratio varied from 0.72 to 1.45. When the resistance factor of 0.75 was added, the mean increased to 1.41 with a standard deviation of 0.15 and a coefficient of variation of 0.11.

The mean by the Eurocode 4 was 0.99 with a standard deviation of 0.12 and a coefficient of variation of 0.12. When partial safety factors of 1.1 for the structural steel, of 1.5 for the concrete, of 1.15 for the reinforcing steel are used, the mean increased to 1.22 with a standard deviation of 0.14 and a coefficient of variance of 0.11. For rectangular concrete filled tube columns, the predictions by AISC 1999, AISC 2005 and Eurocode seem to work well.

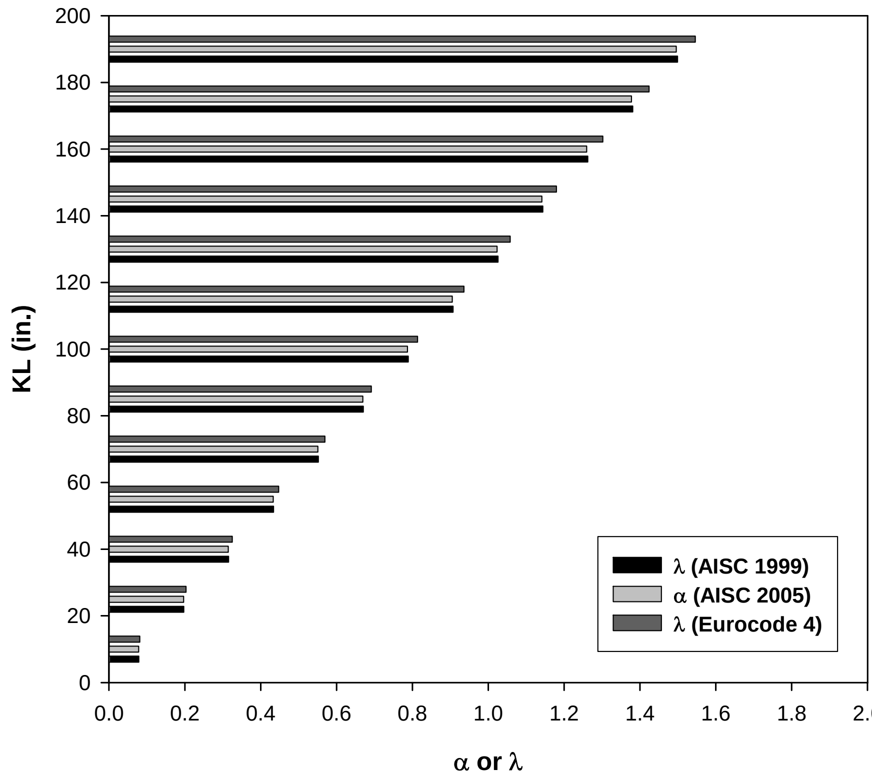


Figure 4-46 Comparison of slenderness ratio for a typical column in the RCFT database (Baba, Fujimoto, Mukai, and Nishiyama, 1995)

Figure 4-46 shows a comparison of slenderness ratios for 45th column in the rectangular CFT column database. From the standpoint of slenderness, the ratio of α/λ was 0.971 for the AISC 2005 vs. AISC 1999 and 0.967 for the AISC 2005 vs. Eurocode 4. For rectangular concrete filled tube columns, the slenderness parameters of λ and α are almost the same values for the AISC 1999, AISC 2005 and Eurocode 4.

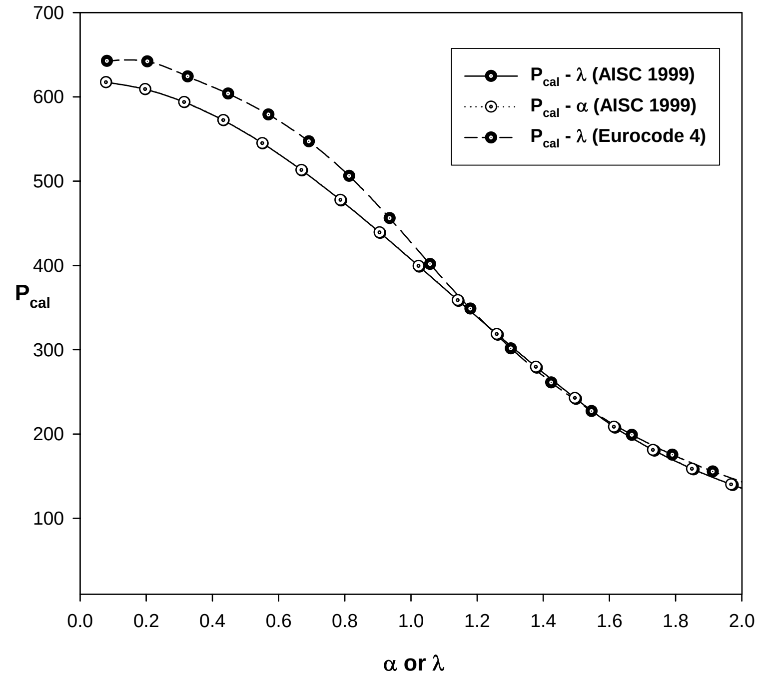


Figure 4-47 Axial capacity vs. slenderness for a typical column in RCFT database (Baba, Fujimoto, Mukai, and Nishiyama, 1995)

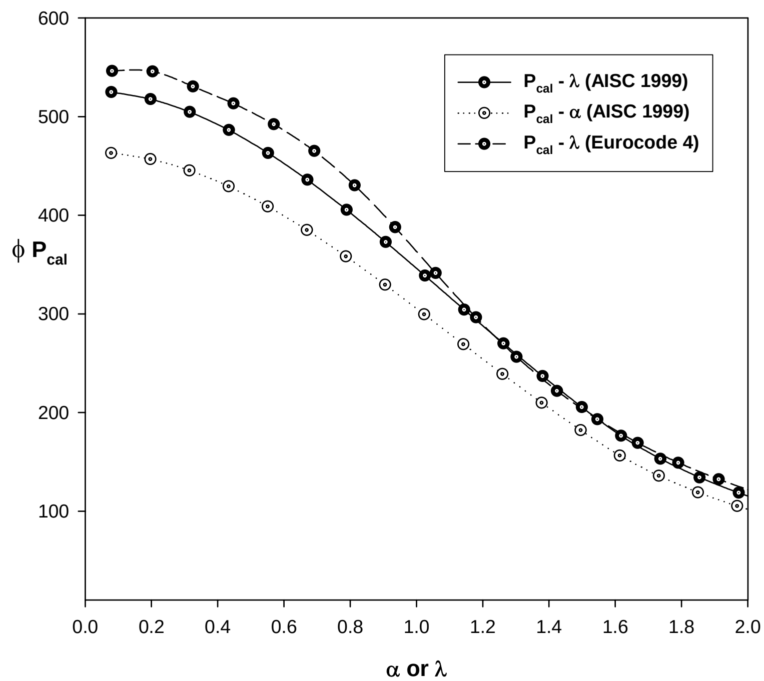


Figure 4-48 Axial capacity vs. slenderness for a typical column in RCFT Database (Baba, Fujimoto, Mukai, and Nishiyama, 1995)

Figures 4-47 and 4-48 show two examples of the strength vs. slenderness curves for a typical specimen in the database (43th column, Baba, Fujimoto, Mukai, and Nishiyama, 1995). As shown in Figure 4-47, the P_{cal} value by AISC 2005 is almost the same as that by the AISC 1999 for all slenderness as the difference is not distinguishable in Figure 4-47. In contrast, when the comparison is made including a resistance factor, the mean value by the AISC 1999 method is larger than that by AISC 2005 method. The mean design load for AISC 2005/1999 is 1.13 as the AISC 1999 graph plots higher than the AISC 2005 curve in Figure 4-48. The P_{cal} value given by Eurocode 4 is larger at a low slenderness ratio.

4.1.6 RCFT Beam-Columns

There were a total of 194 concrete-filled tube beam-columns in the database. From those, 43 CFT beam-columns were eliminated because they failed the local buckling check of either AISC specification or Eurocode. Thirty seven RCFT beam-columns were discarded because they are bi-axially loaded. Thirty-two RCFT beam-columns were not included because they had poor experimental-to-predicted ratio results (Baba, Fujimoto, Mukai, and Nishiyama, 1995). Five tests were reinforced inside the tube (Grauers, 1993). Thirteen tests were not used because of problems during testing (Hardika, Gardner, 2004; Furlong, 1967). Finally, 62 CFT columns were used for analysis and included in the reduced database.

As shown in Figure 4-37, the yield stress ranged from 36.8 ksi to 108.8 ksi. The concrete compressive strength was distributed as shown in Figure 4-38. The maximum compressive strength was 14.9 ksi and the minimum was 4.2 ksi. The structural steel ratio ranged from 11.1% to 24.9% as shown in Figure 4-40. The B/t ratio ranged from 15 to

35.1 as shown in Fig. 4-42. Figure 4.49 shows a 3D plot of the yield stress, concrete compressive strength and reinforcement ratio for beam-columns. This plot emphasizes the large gaps in the database from 8 ksi to 12 ksi in compressive strength of the concrete, more than 70 ksi of yield stress and under 0.1 of structural steel ratio. The ratio for e/D ranged from 0.1 to 4.6 as shown in Fig. 4-50. Figures 4-51, 4-52 and 4-53 show scatter plots of the data by AISC 1999, AISC 2005 and Eurocode 4 versus the slenderness parameter, respectively.

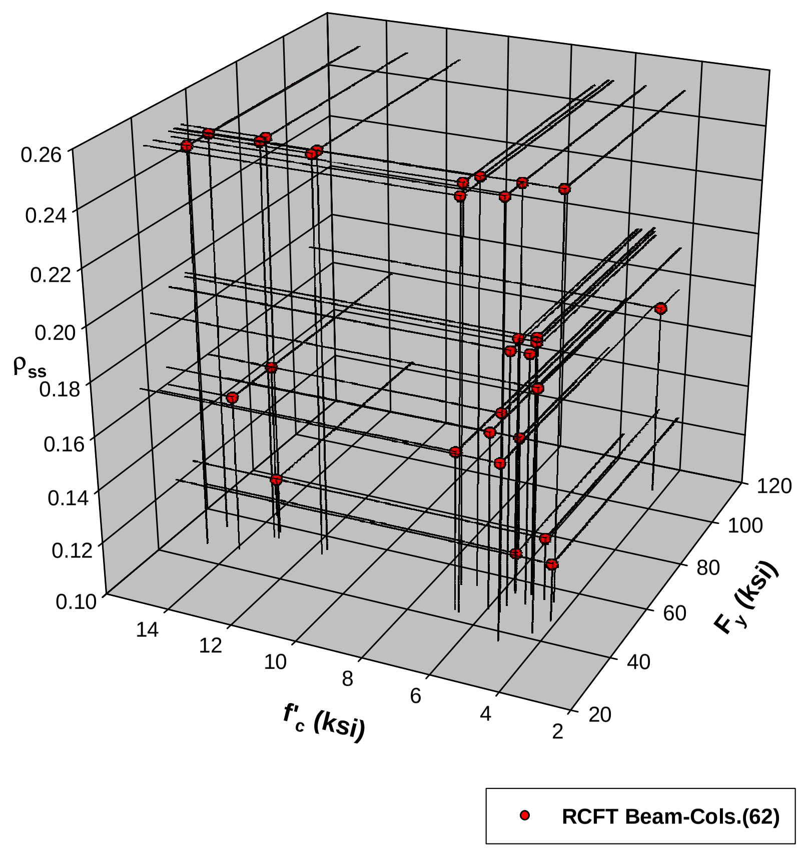


Figure 4-49 Frequency distribution of F_y, f'_c and ρ_{ss} for Reduced RCFT beam-column Database

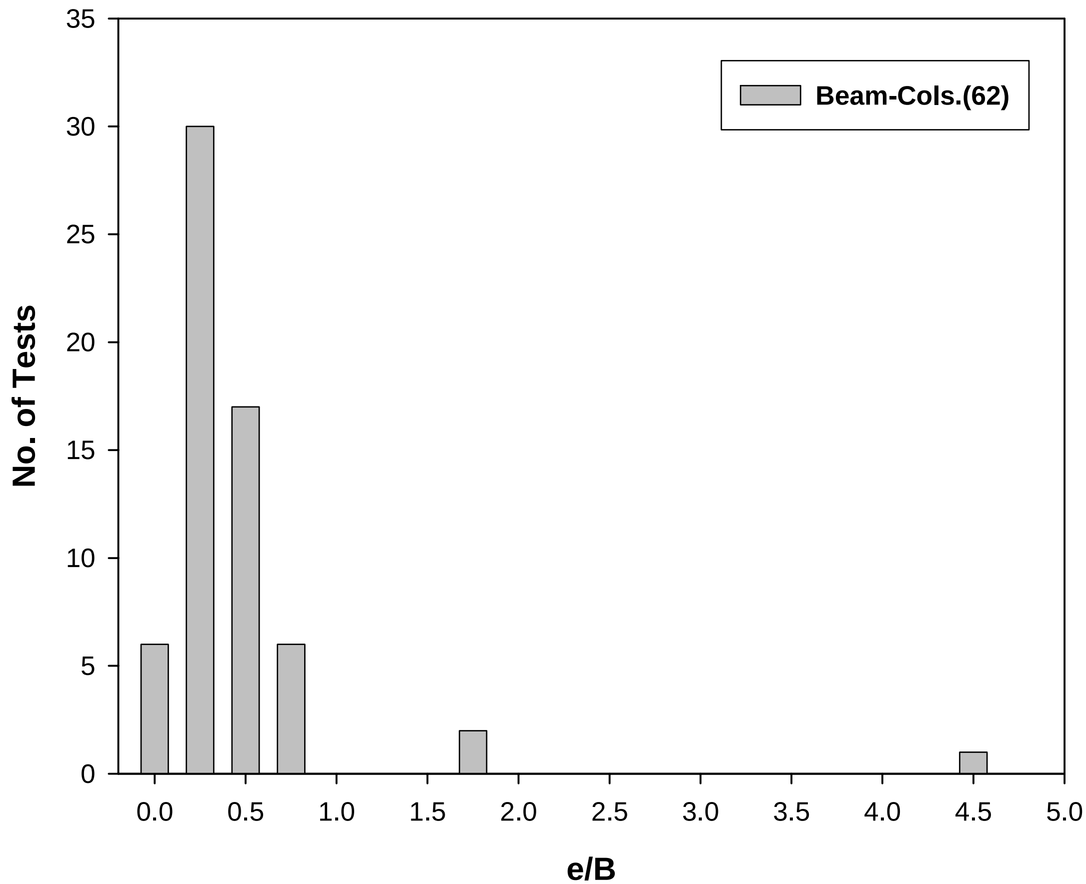


Figure 4-50 Frequency distribution of e/B for Reduced RCFT Database

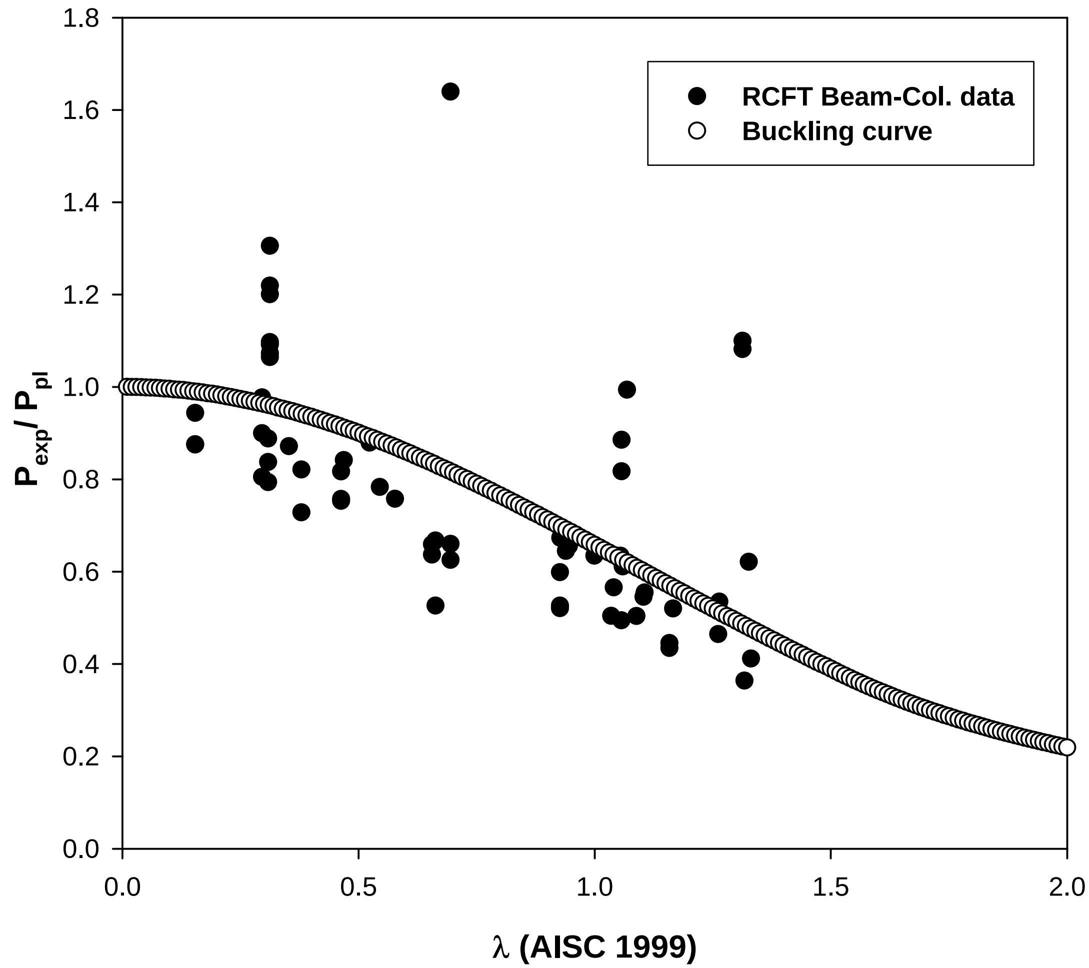


Figure 4-51 P_{exp}/P_{pl} with AISC buckling curve for RCFT beam-columns by AISC 1999

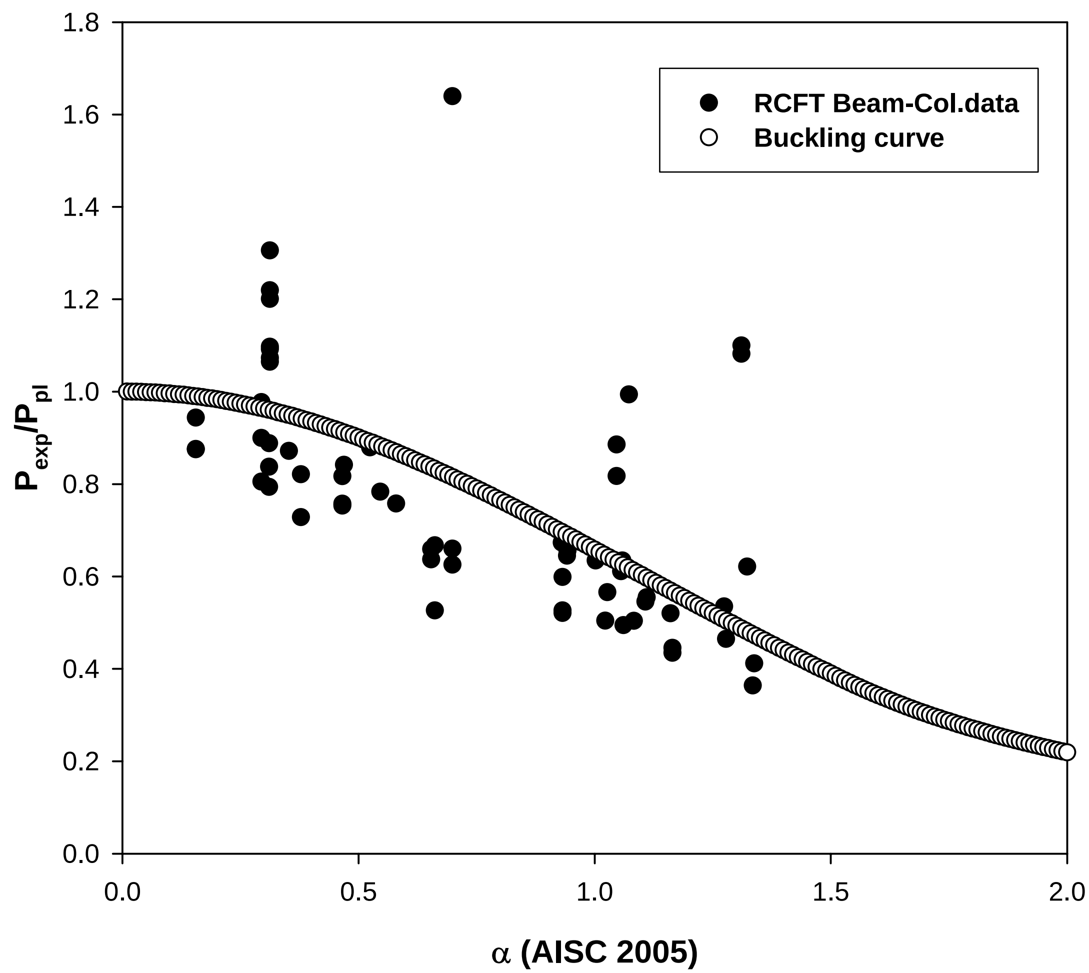


Figure 4-52 P_{exp}/P_{pl} with AISC buckling curve for RCFT beam-columns by AISC 2005

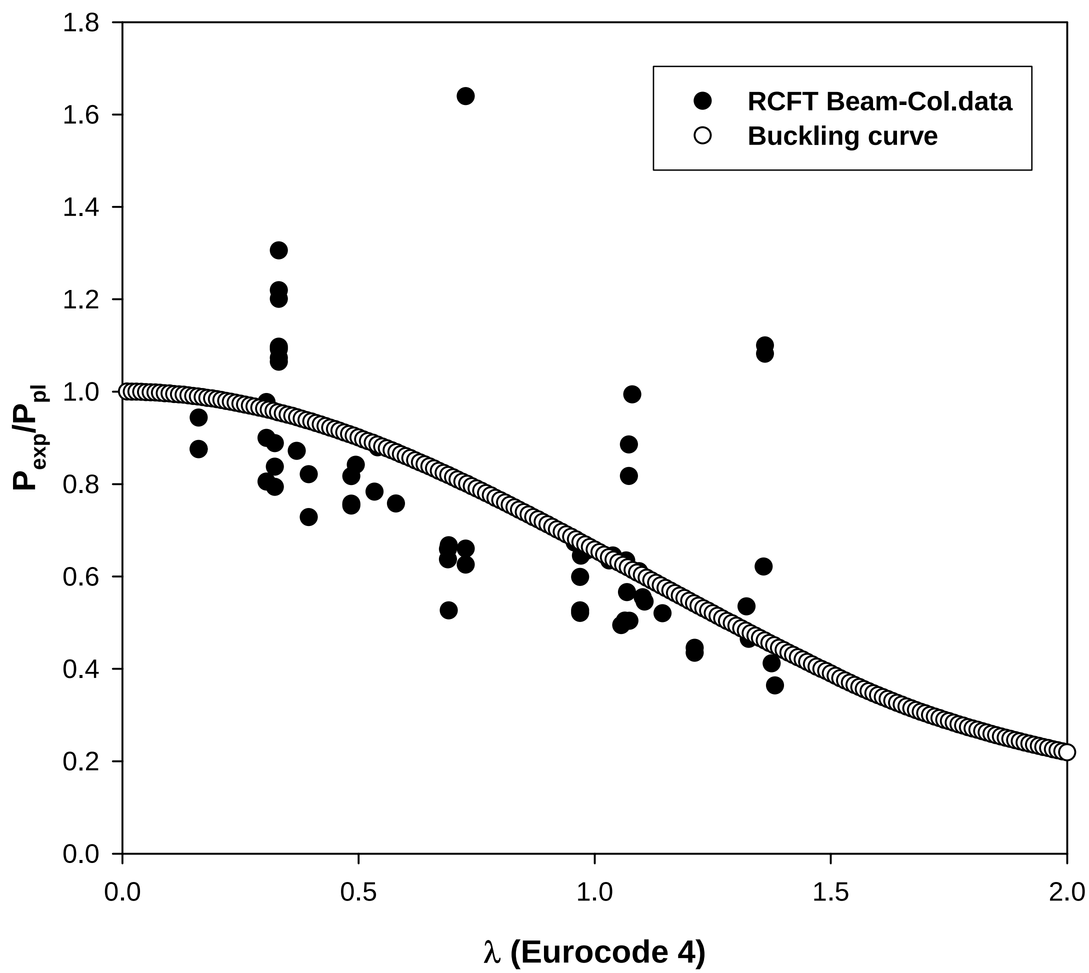


Figure 4-53 P_{exp}/P_{pl} with AISC buckling curve for RCFT beam-columns by Eurocode 4

Table 4-6 Comparison of Beam-column strengths, and α/λ

<table>
<tr><th colspan="2">Type</th><th>No. of Tests</th><th>Mean</th><th>Standard Deviation</th><th>COV</th><th>Mean Design Axial Load(AISC) 2005/1999 (COV)</th></tr>
<tr><td rowspan="5">Rectangular CF Beam-columns (RCFT)</td><td>AISC 1999</td><td rowspan="5">62</td><td>1.27</td><td>0.33</td><td>0.26</td><td rowspan="2">0.87 (0.12)</td></tr>
<tr><td>AISC 2005</td><td>0.99</td><td>0.28</td><td>0.28</td></tr>
<tr><td>Eurocode 4</td><td>1.19</td><td>0.4</td><td>0.34</td><td rowspan="3"></td></tr>
<tr><td>α/λ (AISC 2005/ AISC 1999)</td><td>1.0</td><td>0.01</td><td>0.01</td></tr>
<tr><td>α/λ (AISC 2005/ Eurocode 4)</td><td>0.97</td><td>0.02</td><td>0.02</td></tr>
</table>

F[illegible]nfactored predictions, the mean experimental to calculated axial capacity ratio by the AISC 1999 method was 1.27, the standard deviation was 0.33, and the coefficient of variation was 0.26. The maximum ratio was 2.54 and minimum ratio was 0.83. When a resistance factor of 0.85 for compression and of 0.9 for bending were considered, the mean ratio was changed to 1.42, the standard deviation to 0.36 and the coefficient of variation to 0.25, respectively. By the AISC 2005 method, the mean ratio was 0.99, the standard deviation was 0.28, and the coefficient of variation was 0.28. The maximum ratio was 2.06, and minimum ratio was 0.66. When a resistance factor of 0.75 for compression and of 0.9 for bending were considered, the mean increased to 1.23 with a standard deviation of 0.35 and a coefficient of variation of 0.28. The AISC 1999 method gives comparatively lower values and can be conservative, as shown in Table 4-6. Also, if high strength concrete (>8 ksi) specimens are compared, the ratio by AISC 1999 is 1.33 and the ratio by AISC 2005 is 0.99. This shows that the AISC 1999 method does not work well for high strength concrete.

The mean by the Eurocode was 1.19 with a standard deviation of 0.4 and a coefficient of variation of 0.34. When a partial safety factor of 1.1 for the structural steel, of 1.5 for the concrete, of 1.15 for the reinforcing steel are used, the mean is 1.37 with a standard deviation of 0.43 and a coefficient of variance of 0.31. For high strength concrete (>8 ksi) specimens, the ratio by the Eurocode is 1.17. This shows that the Eurocode 4 predictions also work very well for high strength concrete.

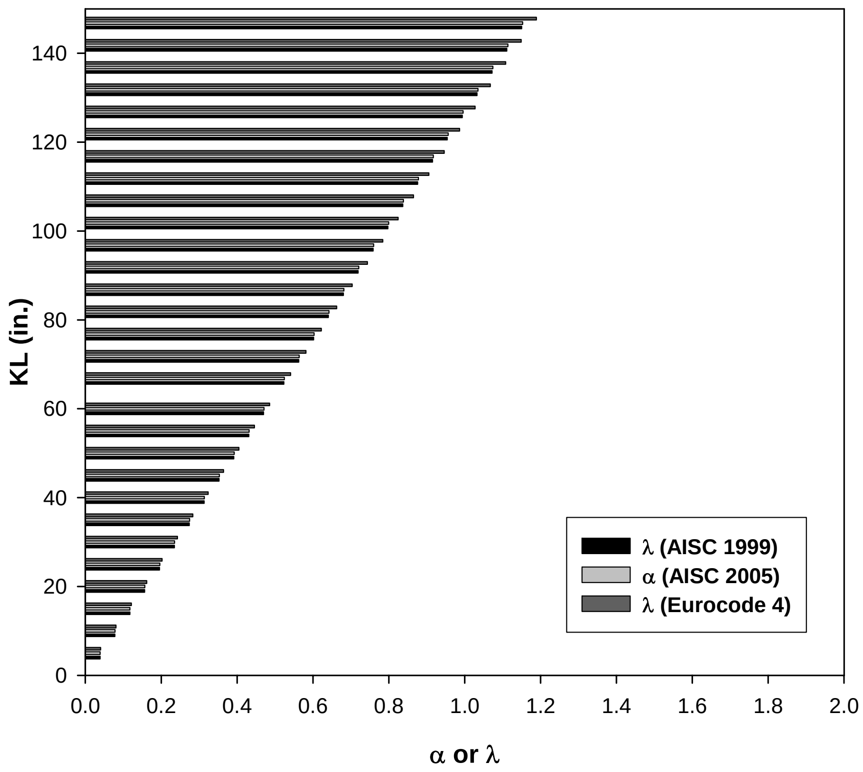


Figure 4-54 Comparison of slenderness ratio for a typical beam-column in the RCFT beam-column database (Grauers, 1993)

Figure 4-54 shows a comparison of slenderness ratios for 29th beam-column in the database of CFT beam-columns given an effective gth. When comparing derness, the ratio of α/λ was 1.0 for the AISC 2005/AISC 1999 and 0.97 for the

AISC 2005/ Eurocode 4. For rectangular concrete filled tube beam-columns, the slenderness parameters λ and α are almost same values at AISC 1999, AISC 2005 and Eurocode 4.

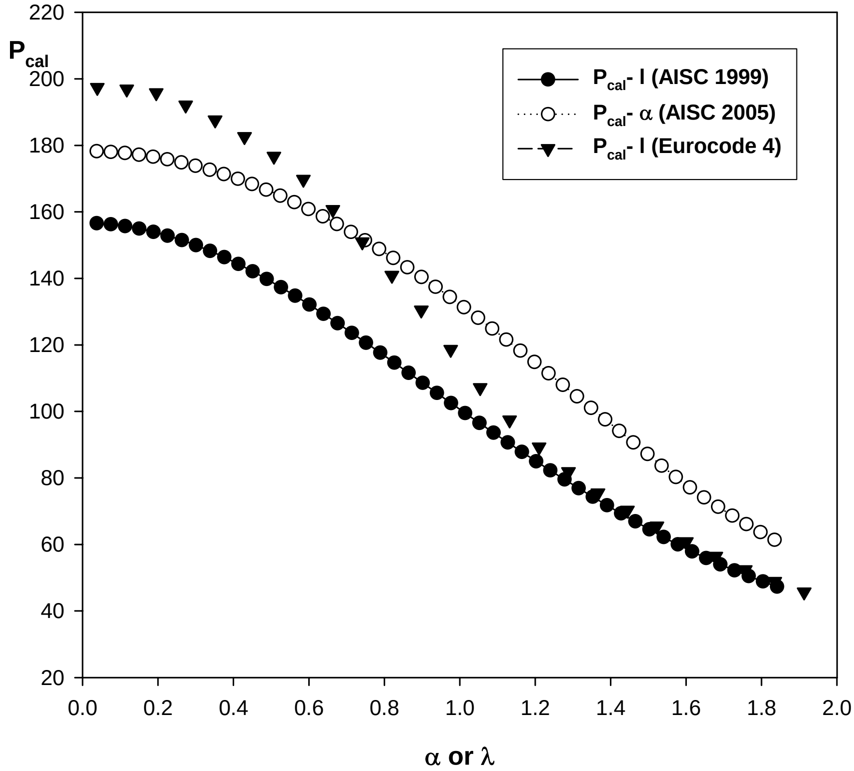


Figure 4-55 Axial capacity vs. slenderness for a typical beam-column in the RCFT beam-column database (Grauers, 1993)

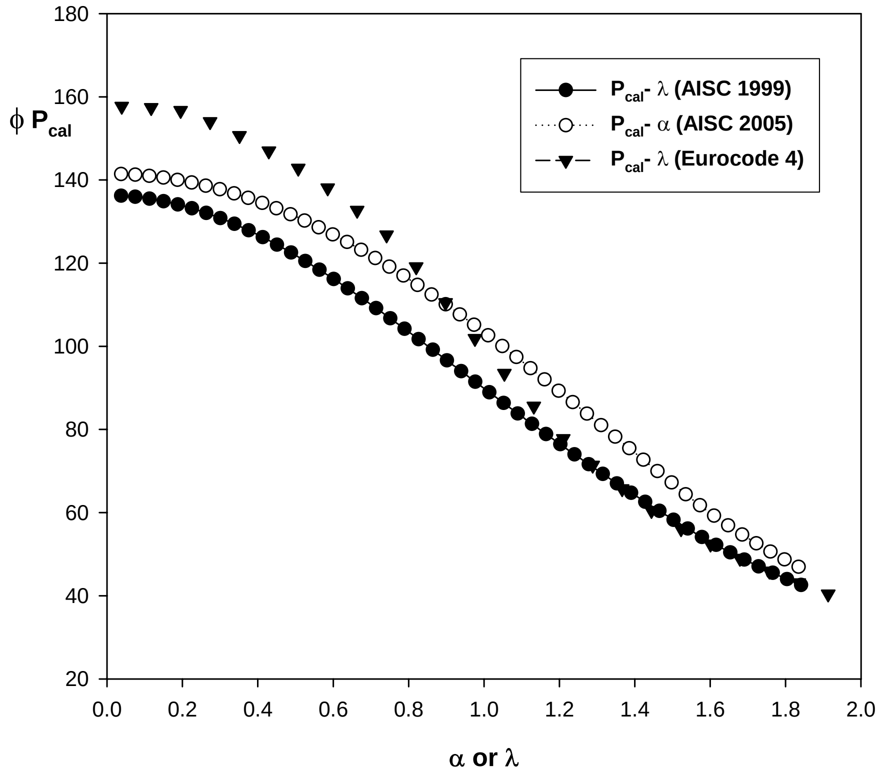


Figure 4-56 Axial capacity vs. slenderness for a typical beam-column in the RCFT beam-column database (Grauers, 1993)

As shown in Figures 4-55 and 4-56, the P_{cal} value by the AISC 2005 method is larger than that by the AISC 1999 method at all slenderness, both with and without resistance factors (29th beam-column, Grauers, 1993). For design, the mean value by the AISC 2005 method is smaller than that by AISC 1999 method. The ratio of AISC 2005/1999 is 0.87, as shown in Table 4-6, and the AISC 2005 data plots higher than the AISC 1999 curve in Figure 4-56. For the Eurocode 4, P_n is a large value at low slenderness.

CHAPTER Ⅴ

CONCLUSIONS

The objectives of this thesis are to augment a database of composite column tests and to utilize the database to evaluate the proposed AISC 2005 provisions. The database was compiled by incorporating as many tests as possible on composite column and beam-column as could be found in the open literature. Data included specimens subjected to concentric and eccentric axial loads, monotonic and cyclic load applications, single and double curvature configurations, and lateral loads. However, only specimens with monotonic loading and single curvature were used in the comparisons.

The database consists of column and beam-column steel-concrete columns (or encased, SRC), circular concrete filled tubes (CCFT), and rectangular concrete filled tube (RCFT). The database includes 119 SRC columns, 136 SRC beam-columns, 312 circular CFT columns, 198 circular CFT beam-columns, 222 rectangular CFT columns and 194 rectangular CFT beam-columns. The database has a total of 1181 specimens; this represents an addition of 451 specimens over those in the Aho database (Aho 1996).

A summary on the database, shown in Table 5-1, lists the total number of tests and the range of material and geometric properties found in those specimens. Table 5-2 shows comparisons of the predicted results by three specifications for the specimens in the reduced database. The Eurocode gives good predictions for columns and the AISC 2005 method performs very well for beam-columns. For rectangular CFT columns, all three methods predict the ultimate capacity very well. The main improvement for the AISC 2005 method is its ability to handle specimens which have high yield stress and/or high strength concrete.

Table 5-1 Database summary

	SRC		**Circular CFT**		**Rect. CFT**	
	Cols.	Beam-Cols.	Cols.	Beam-Cols.	Cols.	Beam-Cols.
Total No. of Tests	119	136	312	198	222	194
No. of Tests for Analysis	89	117	210	118	103	62
Maximum F_y (ksi)	72.7	58.0	121.0	70.0	120.8	108.8
Minimum F_y (ksi)	32.4	32.3	32.1	27.5	36.9	36.8
Maximum f'_c (ksi)	9.5	6.8	16.5	16.3	14.9	14.9
Minimum f'_c (ksi)	1.8	1.8	2.6	2.9	2.6	4.2
Maximum L/r	466.7	247.2	133.8	87.1	91.1	91.2
Maximum ρ_{ss}	12.9%	2.7%	27%	46.6%	26.6%	24.9%
Minimum ρ_{ss}	2.7%	14.6%	5.5%	5.1%	7.1%	11.1%

When comparing design values (i.e., including the resistance factor,) the mean value by the AISC 1999 method is larger than that by AISC 2005 method for SRC, CCFT and RCFT columns. However, the mean value by the AISC 1999 method is smaller than that by AISC 2005 method for SRC, CCFT and RCFT beam-columns.

Table 5-2 Database results

Type		No. of Tests	Mean	Standard Deviation	COV	Mean Design Axial Load(AISC) 2005/1999 (COV)
Encased columns (SRC)	AISC 1999	89	1.22	0.19	0.16	1.11(0.15)
	AISC 2005		1.18	0.20	0.17	
	Eurocode 4		1.09	0.14	0.13	
Encased Beam-columns (SRC)	AISC 1999	117	1.41	0.32	0.23	0.82 (0.23)
	AISC 2005		1.03	0.25	0.24	
	Eurocode 4		1.21	0.22	0.18	
Circular CF columns (CCFT)	AISC 1999	210	1.28	0.19	0.15	1.08 (0.04)
	AISC 2005		1.23	0.18	0.15	
	Eurocode 4		1.06	0.18	0.17	
Circular CF Beam-columns (CCFT)	AISC 1999	118	1.49	0.33	0.22	0.88 (0.21)
	AISC 2005		1.14	0.22	0.19	
	Eurocode 4		1.25	0.19	0.15	
Rectangular CF columns (RCFT)	AISC 1999	103	1.06	0.12	0.11	1.13 (0.01)
	AISC 2005		1.06	0.12	0.11	
	Eurocode 4		0.99	0.12	0.12	
Rectangular CF Beam-columns (RCFT)	AISC 1999	62	1.27	0.33	0.26	0.87 (0.12)
	AISC 2005		0.99	0.28	0.28	
	Eurocode 4		1.19	0.40	0.34	

Table 5-3 Comparison of α/λ

Type		No. of Tests	Mean	Standard Deviation	COV
Encased columns (SRC)	α/λ (AISC 2005/ AISC 1999)	89	1.32	0.18	0.13
	α/λ (AISC 2005/ Eurocode 4)		1.34	0.26	0.19
Encased Beam-columns (SRC)	α/λ (AISC 2005/ AISC 1999)	117	1.47	0.26	0.24
	α/λ (AISC 2005/ Eurocode 4)		1.38	0.21	0.15
Circular CF columns (CCFT)	α/λ (AISC 2005/ AISC 1999)	210	1.03	0.06	0.06
	α/λ (AISC 2005/ Eurocode 4)		0.99	0.05	0.05
Circular CF Beam-columns (CCFT)	α/λ (AISC 2005/ AISC 1999)	118	1.03	0.02	0.02
	α/λ (AISC 2005/ Eurocode 4)		0.99	0.04	0.04
Rectangular CF columns (RCFT)	α/λ (AISC 2005/ AISC 1999)	103	0.99	0.14	0.14
	α/λ (AISC 2005/ Eurocode 4)		0.97	0.13	0.13
Rectangular CF Beam-columns (RCFT)	α/λ (AISC 2005/ AISC 1999)	62	1.0	0.01	0.01
	α/λ (AISC 2005/ Eurocode 4)		0.97	0.02	0.02

When the AISC 2005 and AISC 1999 methods are compared, the value of the slenderness factor α (AISC 2005) is always larger than that of λ (AISC 1999 slenderness factor) for SRC columns and beam-columns. When the AISC 2005 and the Eurocode 4 are compared, the value of α (AISC 2005 factor) is always larger than that of λ (Eurocode factor) for SRC columns and beam-columns. The difference between the two slenderness parameters increases with increasing column slenderness. Thus, AISC 2005 provides larger resistances with increasing slenderness than other specifications for both SRC columns and beam-columns. For both CCFT and RCFT columns and beam-columns, α/λ has almost same values for both AISC and Eurocode.

APPENDIX A

COMPOSITE COLUMN DATABASE

Table A-1 - SRC Column Database

Col. No.	Spec. No.	Fy (ksi)	f'c (ksi)	Fyr (ksi)	Steel Section	Ac (in^2)	As (in^2)	Ar (in^2)	h1 (in)	h2 (in)	kl (in)	Pexp (k)	Pexp/Ppred by AISC 1999	AISC 2005	Eurocode 4	Plastic
Janss and Anslijn, 1974																
1	1.1	41.3	5.49	60.0	HEB 140	81.94	6.66	0.70	9.45	9.45	168.5	482.7	1.21	1.40	1.11	0.72
2	1.2	41.3	5.08	60.0	HEB 140	81.94	6.66	0.70	9.45	9.45	168.5	489.3	1.26	1.46	1.19	0.76
3	1.3	39.6	4.86	60.0	HEB 140	81.94	6.66	0.70	9.45	9.45	168.3	469.5	1.25	1.43	1.18	0.76
4	2.1	42.4	4.86	60.0	HEB 140	81.94	6.66	0.70	9.45	9.45	137.2	526.8	1.23	1.28	1.13	0.83
5	2.2	42.4	4.23	60.0	HEB 140	81.94	6.66	0.70	9.45	9.45	136.7	489.3	1.20	1.25	1.13	0.82
6	2.3	42.4	5.08	60.0	HEB 140	81.94	6.66	0.70	9.45	9.45	136.9	579.7	1.33	1.38	1.21	0.89
7	3.1	40.0	5.44	60.0	HEB 140	81.94	6.66	0.70	9.45	9.45	98.0	590.7	1.22	1.12	1.04	0.89
8	3.2	40.0	5.49	60.0	HEB 140	81.94	6.66	0.70	9.45	9.45	97.5	502.5	1.03	0.94	0.88	0.76
9	3.3	40.0	4.86	60.0	HEB 140	81.94	6.66	0.70	9.45	9.45	98.0	526.8	1.14	1.05	1.00	0.85
10	4.1	40.0	4.86	60.0	HEB 140	81.94	6.66	0.70	9.45	9.45	50.7	573.0	1.15	0.98	0.95	0.92
11	4.2	40.0	4.23	60.0	HEB 140	81.94	6.66	0.70	9.45	9.45	50.5	555.4	1.18	1.02	0.99	0.96
12	4.3	40.0	5.12	60.0	HEB 140	81.94	6.66	0.70	9.45	9.45	49.3	617.1	1.21	1.02	0.99	0.97
13	5.1	54.9	4.53	60.0	HEB 140	81.94	6.66	0.70	9.45	9.45	137.4	529.0	1.13	1.23	1.08	0.76
14	5.2	54.9	4.88	60.0	HEB 140	81.94	6.66	0.70	9.45	9.45	137.1	590.7	1.23	1.34	1.16	0.82
15	5.3	54.9	4.63	60.0	HEB 140	81.94	6.66	0.70	9.45	9.45	137.2	555.4	1.17	1.28	1.12	0.79
16	6.1	72.7	4.53	60.0	HEB 140	81.94	6.66	0.70	9.45	9.45	168.3	529.0	1.13	1.52	1.18	0.65
17	6.2	72.7	4.88	60.0	HEB 140	81.94	6.66	0.70	9.45	9.45	168.3	484.9	1.02	1.36	1.03	0.58
18	6.3	72.0	4.64	60.0	HEB 140	81.94	6.66	0.70	9.45	9.45	168.3	557.6	1.19	1.59	1.23	0.68
19	7.1	70.7	4.61	60.0	HEB 140	81.94	6.66	0.70	9.45	9.45	137.4	555.4	1.04	1.20	1.03	0.69
20	7.2	70.7	4.89	60.0	HEB 140	81.94	6.66	0.70	9.45	9.45	137.3	588.5	1.08	1.25	1.06	0.71
21	7.3	70.7	4.64	60.0	HEB 140	81.94	6.66	0.70	9.45	9.45	137.3	577.4	1.08	1.24	1.06	0.71
22	8.1	72.4	4.86	60.0	HEB 140	81.94	6.66	0.70	9.45	9.45	97.8	546.6	0.86	0.86	0.76	0.65
23	8.2	72.4	5.49	60.0	HEB 140	81.94	6.66	0.70	9.45	9.45	97.8	531.2	0.81	0.81	0.70	0.60
24	8.3	72.4	5.44	60.0	HEB 140	81.94	6.66	0.70	9.45	9.45	98.0	573.0	0.88	0.88	0.76	0.65
25	9.1	39.5	4.23	60.0	IPE 220	98.32	5.18	0.70	12.60	8.27	137.3	513.5	1.20	1.05	1.08	0.86
26	9.2	39.5	5.44	60.0	IPE 220	98.32	5.18	0.70	12.60	8.27	137.3	568.6	1.17	1.02	1.03	0.82
27	9.3	39.5	4.89	60.0	IPE 220	98.32	5.18	0.70	12.60	8.27	137.2	462.8	1.00	0.88	0.89	0.71
28	10.1	70.7	4.86	60.0	IPE 220	98.32	5.18	0.70	12.60	8.27	137.2	517.9	0.89	0.83	0.84	0.64
29	10.2	70.7	4.50	60.0	IPE 220	98.32	5.18	0.70	12.60	8.27	97.8	608.3	0.99	0.88	0.90	0.78
30	10.3	71.3	4.63	60.0	IPE 220	98.32	5.18	0.70	12.60	8.27	137.1	531.2	0.92	0.86	0.88	0.66

Table A-1 - SRC Column Database

Col. No.	Spec. No.	Fy (ksi)	f'c (ksi)	Fyr (ksi)	Steel Section	Ac (in^2)	As (in^2)	Ar (in^2)	h1 (in)	h2 (in)	kl (in)	Pexp (k)	Pexp/Ppred by AISC 1999	AISC 2005	Eurocode 4	Plastic
Stevens, 1965																
31	B1	40.6	2.56	40.0	BS 3 x 1.5	16.32	1.18	0.00	5.00	3.50	46.0	81.4	1.19	1.10	1.10	0.98
32	B2	40.6	1.84	40.0	BS 3 x 1.5	16.32	1.18	0.00	5.00	3.50	64.0	60.1	1.03	1.02	1.02	0.82
33	B3	40.6	2.24	40.0	BS 3 x 1.5	16.32	1.18	0.00	5.00	3.50	82.0	63.0	1.10	1.16	1.17	0.80
34	B4	40.6	2.00	40.0	BS 3 x 1.5	16.32	1.18	0.00	5.00	3.50	100.0	43.6	0.86	0.99	1.02	0.58
35	B5	40.6	2.52	40.0	BS 3 x 1.5	16.32	1.18	0.00	5.00	3.50	118.0	50.6	1.07	1.37	1.36	0.61
36	B6	40.6	2.20	40.0	BS 3 x 1.5	16.32	1.18	0.00	5.00	3.50	136.0	36.1	0.90	1.29	1.26	0.46
37	B7	40.6	2.76	40.0	BS 3 x 1.5	16.32	1.18	0.00	5.00	3.50	154.0	33.9	0.95	1.52	1.34	0.39
38	A1	41.1	2.32	40.0	BS 5 x 4.5	39.62	5.88	0.00	7.00	6.50	13.0	352.0	1.19	1.10	1.10	1.10
39	A2	41.4	2.08	40.0	BS 5 x 4.5	39.62	5.88	0.00	7.00	6.50	46.0	308.0	1.08	1.02	1.01	0.98
40	A5	41.4	2.32	40.0	BS 5 x 4.5	39.62	5.88	0.00	7.00	6.50	118.0	288.0	1.18	1.12	1.14	0.90
41	A6	41.4	2.48	40.0	BS 5 x 4.5	39.62	5.88	0.00	7.00	6.50	153.0	231.0	1.07	1.03	1.07	0.71
42	A3	41.4	2.28	40.0	BS 5 x 4.5	39.62	5.88	0.00	7.00	6.50	82.0	316.8	1.17	1.10	1.11	0.99
43	A4	41.4	2.66	40.0	BS 5 x 4.5	39.62	5.88	0.00	7.00	6.50	82.0	297.0	1.07	0.99	1.00	0.89
44	FA1	32.4	2.28	40.0	BS 12 x 8	172.90	19.10	0.00	16.00	12.00	36.0	1051.0	1.23	1.11	1.10	1.10
45	FA2	32.4	2.44	40.0	BS 12 x 8	172.90	19.10	0.00	16.00	12.00	72.0	990.0	1.15	1.03	1.01	1.01
46	FA3	32.4	2.16	40.0	BS 12 x 8	172.90	19.10	0.00	16.00	12.00	108.0	926.0	1.13	1.02	1.01	0.99
47	FA4	32.4	2.40	40.0	BS 12 x 8	172.90	19.10	0.00	16.00	12.00	144.0	937.0	1.14	1.02	1.02	0.96
48	FA5	32.4	2.40	40.0	BS 12 x 8	172.90	19.10	0.00	16.00	12.00	180.0	933.0	1.16	1.04	1.05	0.96
49	S1G	33.3	2.16	40.0	BS 8 x 6	69.70	10.30	0.00	10.00	8.00	84.0	528.0	1.27	1.17	1.16	1.12
50	S1S	33.3	2.72	40.0	BS 8 x 6	69.70	10.30	0.00	10.00	8.00	84.0	567.6	1.30	1.18	1.17	1.13
51	S2G	33.3	3.44	40.0	BS 8 x 6	109.50	10.30	0.20	12.00	10.00	84.0	638.0	1.15	1.00	0.99	0.95
52	S2S	33.3	3.48	40.0	BS 8 x 6	109.50	10.30	0.20	12.00	10.00	84.0	704.0	1.27	1.10	1.09	1.04
53	S3G	33.3	2.68	40.0	BS 8 x 6	157.50	10.30	0.20	14.00	12.00	84.0	801.0	1.37	1.18	1.16	1.13
54	S3S	33.3	3.40	40.0	BS 8 x 6	157.50	10.30	0.20	14.00	12.00	84.0	930.6	1.43	1.22	1.19	1.15
55	RE1a	42.3	2.44	40.0	BS 5 x 4.5	39.62	5.88	0.00	7.00	6.50	118.0	294.8	1.18	1.12	1.14	0.89
56	RE1b	42.3	2.20	40.0	BS 5 x 4.5	39.62	5.88	0.00	7.00	6.50	118.0	275.0	1.12	1.06	1.09	0.85
57	RE2a	42.3	2.32	40.0	BS 5 x 4.5	39.62	5.88	0.00	7.00	6.50	118.0	270.6	1.09	1.04	1.06	0.83
58	RE2b	42.3	2.76	40.0	BS 5 x 4.5	39.62	5.88	0.00	7.00	6.50	118.0	264.0	1.04	0.97	0.99	0.77
59	RE3a	42.3	2.68	40.0	BS 5 x 4.5	39.42	5.88	0.20	7.00	6.50	118.0	308.0	1.20	1.12	1.13	0.89
60	RE3b	42.3	2.32	40.0	BS 5 x 4.5	39.42	5.88	0.20	7.00	6.50	118.0	272.8	1.09	1.02	1.04	0.82
61	RE4a	42.3	2.40	40.0	BS 5 x 4.5	39.62	5.88	0.00	7.00	6.50	118.0	266.2	1.07	1.01	1.03	0.81
62	RE4b	42.3	2.24	40.0	BS 5 x 4.5	39.62	5.88	0.00	7.00	6.50	118.0	279.4	1.14	1.08	1.10	0.86

Table A-1 - SRC Column Database

Col. No.	Spec. No.	Fy (ksi)	f'c (ksi)	Fyr (ksi)	Steel Section	Ac (in^2)	As (in^2)	Ar (in^2)	h1 (in)	h2 (in)	kl (in)	Pexp (k)	Pexp/Ppred by AISC 1999	AISC 2005	Eurocode 4	Plastic
Astaneh-Asl, Chen, and Moehle, 1992																
63		57.3	6.25	70.0	W8 x 40	181.69	11.91	2.40	14.00	14.00	68.0	1898.0	1.56	1.12	1.37	1.05
64		57.3	6.25	70.0	W8 x 40	181.69	11.91	2.40	14.00	14.00	68.0	1900.0	1.56	1.12	1.37	1.05
65		57.3	8.20	70.0	W8 x 40	181.69	11.91	2.40	14.00	14.00	68.0	1799.0	1.29	0.91	1.13	0.85
66		57.3	8.20	70.0	W8 x 40	181.69	11.91	2.40	14.00	14.00	68.0	1946.0	1.40	0.99	1.22	0.92
67		57.3	8.20	70.0	W8 x 40	182.85	11.91	1.24	14.00	14.00	68.0	1897.0	1.41	1.00	1.24	0.93
68		57.3	8.20	70.0	W8 x 40	181.69	11.91	2.40	14.00	14.00	68.0	1897.0	1.36	0.96	1.19	0.90
69	S1	53.9	9.52	68.3	W8 x 40	181.93	11.67	2.40	14.00	14.00	180.0	1898.0	1.62	1.39	1.33	0.84
70	S2	53.9	8.92	68.3	W8 x 40	181.93	11.67	2.40	14.00	14.00	180.0	1499.0	1.32	1.13	1.10	0.69
71	S3	53.9	9.10	84.9	W8 x 40	183.09	11.67	1.24	14.00	14.00	180.0	1523.0	1.34	1.19	1.13	0.71
72	S4	53.9	9.38	68.3	W8 x 40	181.93	11.67	2.40	14.00	14.00	180.0	1922.0	1.65	1.42	1.36	0.86
73	S5	55.2	9.49	68.3	W8 x 67	174.23	19.37	2.40	14.00	14.00	180.0	2197.0	1.49	1.27	1.30	0.83
74	S6	50.0	9.35	68.3	W8 x 28	185.35	8.25	2.40	14.00	14.00	180.0	1847.0	1.82	1.64	1.45	0.90
75	S7	55.2	4.34	68.3	W8 x 67	174.23	19.37	2.40	14.00	14.00	180.0	1473.0	1.27	1.14	1.23	0.79
Loke, 1968																
76	1	42.4	3.75		BS 4 x 3	53.17	2.83	0.00	8.00	7.00	89.0	273.5	1.51	1.22	1.44	0.94
77	4	40.7	3.86		BS 4 x 3	53.11	2.89	0.00	8.00	7.00	89.0	264.3	1.45	1.17	1.39	0.91
78	8	39.4	4.14		BS 4 x 3	53.11	2.89	0.00	8.00	7.00	125.0	290.0	1.75	1.60	1.73	0.96
79	13	43.0	3.08		BS 4 x 1.75	54.49	1.51	0.00	8.00	7.00	89.0	180.0	1.45	1.19	1.37	0.87
80	16	39.3	4.57		BS 4 x 3	53.11	2.89	0.00	8.00	7.00	89.0	303.0	1.55	1.24	1.47	0.95
Roik and Schwalbenhofer																
81	V113	45.3	5.82		HE 160 A	111.69	6.01	3.82	11.02	11.02	118.1	1031.9	1.74	1.26	1.42	0.98
Han et al., 1992																
82		33.6	3.0	40.0	H-100x75x3.2x4.5	37.74	1.50	0.44	6.30	6.30	19.7	177.5	1.61	1.10	1.41	1.08
83		33.6	3.0	40.0	H-100x75x3.2x4.5	37.74	1.50	0.44	6.30	6.30	19.7	236.0	2.14	1.46	1.88	1.43
84		33.6	3.0	40.0	H-100x75x3.2x4.5	37.74	1.50	0.44	6.30	6.30	19.7	251.7	2.28	1.56	2.00	1.53
85		44.8	3.1	40.0	H-100x100x6x8	35.84	3.39	0.44	6.30	6.30	19.7	293.0	1.50	1.12	1.35	1.10
86		44.8	3.1	40.0	H-100x100x6x8	35.84	3.39	0.44	6.30	6.30	19.7	353.3	1.80	1.35	1.62	1.33
87		44.8	3.1	40.0	H-100x100x6x8	35.84	3.39	0.44	6.30	6.30	19.7	361.3	1.85	1.38	1.66	1.36

Table A-1 - SRC Column Database

Col. No.	Spec. No.	Fy (ksi)	f'c (ksi)	Fyr (ksi)	Steel Section	Ac (in^2)	As (in^2)	Ar (in^2)	h1 (in)	h2 (in)	kl (in)	Pexp (k)	Pexp/Ppred by AISC 1999	AISC 2005	Eurocode 4	Plastic
Han and Kim, 1995																
88		45.8	3.1	41.0	H-100x100x6x8	35.84	3.39	0.44	6.30	6.30	27.6	353.3	1.53	1.36	1.33	1.31
89		45.8	3.1	42.0	H-100x100x6x8	35.84	3.39	0.44	6.30	6.30	47.2	295.3	1.32	1.22	1.18	1.10
90		46.0	3.0	43.0	H-100x75x3.2x4.5	37.74	1.50	0.44	6.30	6.30	66.9	240.7	1.82	1.75	1.50	1.31
91		45.8	3.1	44.0	H-100x100x6x8	35.84	3.39	0.44	6.30	6.30	66.9	296.6	1.40	1.37	1.29	1.10
92		45.8	3.1	45.0	H-100x100x6x8	35.84	3.39	0.44	6.30	6.30	86.6	292.4	1.48	1.57	1.41	1.09
Light Weight Concrete Specimens																
Janss and Pirapez, 1974																
1	1	46.8	4.25	60	IPE 220	98.32	5.18	0.70	12.60	8.3	136.9	606.1	1.37	1.20	1.15	0.97
2	2	41.4	4.25	60	HEB 140	81.94	6.66	0.70	9.45	9.5	136.9	517.9	1.27	1.45	1.10	0.86
3	3	46.8	4.25	60	IPE 220	98.32	5.18	0.70	12.60	8.3	50.2	590.7	1.17	0.97	0.95	0.94
4	4	41.4	4.25	60	HEB 140	81.94	6.66	0.70	9.45	9.5	50.2	522.3	1.07	0.93	0.89	0.87
5	5	46.8	4.61	60	IPE 220	98.32	5.18	0.70	12.60	8.3	136.9	617.1	1.34	1.18	1.12	0.94
6	6	41.4	4.61	60	HEB 140	81.94	6.66	0.70	9.45	9.5	136.9	537.8	1.28	1.47	1.09	0.86
7	7	46.8	4.61	60	IPE 220	98.32	5.18	0.70	12.60	8.3	50.2	645.8	1.23	1.02	0.99	0.98
8	8	41.4	4.61	60	HEB 140	81.94	6.66	0.70	9.45	9.5	50.2	544.4	1.07	0.94	0.89	0.87
9	9	46.8	4.91	60	IPE 220	98.32	5.18	0.70	12.60	8.3	136.9	427.6	0.90	0.79	0.75	0.63
10	10	41.4	4.91	60	HEB 140	81.94	6.66	0.70	9.45	9.5	136.9	480.5	1.12	1.28	0.94	0.74
11	11	46.8	4.91	60	IPE 220	98.32	5.18	0.70	12.60	8.3	50.2	460.6	0.85	0.70	0.68	0.68
12	12	41.4	4.91	60	HEB 140	81.94	6.66	0.70	9.45	9.5	50.2	502.5	0.97	0.84	0.79	0.78
13	13	46.8	4.47	60	IPE 220	98.32	5.18	0.70	12.60	8.3	168.4	418.8	1.00	0.91	0.84	0.65
14	14	41.4	4.47	60	HEB 140	81.94	6.66	0.70	9.45	9.5	168.4	403.3	1.09	1.46	0.95	0.66
15	15	46.8	4.56	60	IPE 220	98.32	5.18	0.70	12.60	8.3	168.4	440.8	1.04	0.95	0.88	0.68
16	16	41.4	4.56	60	HEB 140	81.94	6.66	0.70	9.45	9.5	168.4	533.4	1.43	1.92	1.24	0.86
17	17	46.8	4.51	60	IPE 220	98.32	5.18	0.70	12.60	8.3	168.4	436.4	1.03	0.95	0.87	0.67
18	18	41.4	4.51	60	HEB 140	81.94	6.66	0.70	9.45	9.5	168.4	471.7	1.27	1.70	1.11	0.76
19	19	46.8	4.27	60	IPE 220	98.32	5.18	0.70	12.60	8.3	97.5	575.2	1.20	1.02	1.00	0.92
20	21	41.4	4.27	60	HEB 140	81.94	6.66	0.70	9.45	9.5	97.5	573.0	1.26	1.24	1.08	0.95
21	23	46.8	4.13	60	IPE 220	98.32	5.18	0.70	12.60	8.3	97.5	599.5	1.27	1.09	1.06	0.97
22	25	41.4	4.13	60	HEB 140	81.94	6.66	0.70	9.45	9.5	97.5	546.6	1.22	1.20	1.05	0.92
23	27	46.8	3.75	60	IPE 220	98.32	5.18	0.70	12.60	8.3	97.5	551.0	1.22	1.05	1.02	0.94
24	29	41.4	3.75	60	HEB 140	81.94	6.66	0.70	9.45	9.5	97.5	447.4	1.04	1.02	0.90	0.79

Table A-1 - SRC Column Database

Col. No.	Spec. No.	Fy (ksi)	f'c (ksi)	Fyr (ksi)	Steel Section	Ac (in^2)	As (in^2)		Ar (in^2)	h1 (in)	h2 (in)	kl (in)	Pexp (k)		Pexp/Ppred by AISC 1999	AISC 2005	Eurocode 4	Plastic
Johnson and May, 1978																		
1	RC1	41.8	4.23	60	152 x 152 UC23	57.38	4.62		7.87	7.87	63.5	301.2	0.88		1.33	1.00	1.09	0.98
2	RC2	41.8	3.51	60	152 x 152 UC23	57.38	4.62		7.87	7.87	63.5	285.5	1.24/0.62					
3	RC3	41.8	3.41	60	152 x 152 UC23	57.38	4.62		7.87	7.87	63.5	305.7	1.07		1.53	1.18	1.31	1.17
4	RC4	41.8	5.01	60	152 x 152 UC23	57.38	4.62		7.87	7.87	116.7	191.1	1.55		1.13	0.74	0.90	0.67
5	RC5	41.8	4.22	60	152 x 152 UC23	57.38	4.62		7.87	7.87	112.6	185.5		0.79	1.10	0.86	1.01	0.60
Stevens, 1965																		
6	FE3	32.3	2.52	40	BS 12 x 8	172.09	19.12	0.79	16.00	12.00	180.0	661.4		1.00	1.22	1.19	1.41	0.84
7	FE4	32.3	2.36	40	BS 12 x 8	172.09	19.12	0.79	16.00	12.00	180.0	478.4		2.00	1.19	1.08	1.29	1.00
8	FE5	32.3	3.92	40	BS 12 x 8	172.09	19.12	0.79	16.00	12.00	180.0	507.1		2.00	1.12	0.92	1.18	0.67
9	FE6	32.3	2.68	40	BS 12 x 8	172.09	19.12	0.79	16.00	12.00	180.0	354.9		3.00	1.06	0.90	1.13	0.83
10	FE7	32.3	2.68	40	BS 12 x 8	172.09	19.12	0.79	16.00	12.00	180.0	291.0		4.00	1.04	0.85	1.12	0.81
11	FE8	32.3	2.80	40	BS 12 x 8	172.09	19.12	0.79	16.00	12.00	180.0	257.9		5.00	1.06	0.84	1.14	0.84
12	FE9	32.3	2.72	40	BS 12 x 8	172.09	19.12	0.79	16.00	12.00	180.0	227.1		6.00	1.06	0.84	1.17	0.87
13	FE10	32.3	3.08	40	BS 12 x 8	172.09	19.12	0.79	16.00	12.00	180.0	196.2		7.00	1.00	0.75	1.14	0.80
14	FE11	32.3	3.00	40	BS 12 x 8	172.09	19.12	0.79	16.00	12.00	180.0	165.3		8.00	0.94	0.70	1.11	0.77
15	CV3	33.6	2.32	40	BS 5 x 4.5	39.62	5.88		7.00	6.50	82.0	158.7		0.75	1.19	1.10	1.35	0.85
16	CV4	33.6	2.96	40	BS 5 x 4.5	39.62	5.88		7.00	6.50	82.0	176.4		0.75	1.24	1.11	1.36	0.85
17	CV5	33.6	3.56	40	BS 5 x 4.5	39.62	5.88		7.00	6.50	82.0	198.4		0.75	1.34	1.16	1.39	0.89
18	CV6	33.6	4.20	40	BS 5 x 4.5	39.62	5.88		7.00	6.50	82.0	224.9		0.80	1.49	1.25	1.48	0.95
19	AE1	33.6	2.48	40	BS 5 x 4.5	39.62	5.88		7.00	6.50	28.6	163.1		1.00	1.26	1.08	1.33	1.21
20	AE7	33.6	2.52	40	BS 5 x 4.5	39.62	5.88		7.00	6.50	45.5	220.5		0.50	1.30	1.16	1.18	1.02
21	AE2	33.6	3.16	40	BS 5 x 4.5	39.62	5.88		7.00	6.50	45.5	160.9		1.00	1.18	0.98	1.08	0.86
22	AE3	33.6	3.04	40	BS 5 x 4.5	39.62	5.88		7.00	6.50	82.0	138.9		1.00	1.10	0.95	1.20	0.75
23	AE8	33.6	2.60	40	BS 5 x 4.5	39.62	5.88		7.00	6.50	118.0	158.7		0.50	1.11	1.15	1.48	0.73
24	AE4	33.6	3.40	40	BS 5 x 4.5	39.62	5.88		7.00	6.50	118.0	116.8		1.00	0.99	0.92	1.25	0.61
25	AE5	33.6	2.76	40	BS 5 x 4.5	39.62	5.88		7.00	6.50	153.0	97.0		1.00	0.97	1.05	1.48	0.55
26	AE9	33.6	1.84	40	BS 5 x 4.5	39.62	5.88		7.00	6.50	153.0	77.2		1.50	1.04	1.10	1.61	0.67
27	AE10	33.6	2.32	40	BS 5 x 4.5	39.62	5.88		7.00	6.50	153.0	72.8		2.00	1.07	1.06	1.49	0.77

Table A-2 - SRC Beam-Column Database

Col. No.	Spec. No.	Fy (ksi)	f'c (ksi)	Fyr (ksi)	Steel Section	Ac (in^2)	As (in^2)	Ar (in^2)	h1 (in)	h2 (in)	kl (in)	Pexp (k)	ey (in)	ex (in)	Pexp/Ppred by AISC 1999	AISC 2005	Eurocode 4	Plastic
Janss and Anslijn, 1974																		
28	11.1	41.5	4.98	40	HEB 140	82.21	6.39	0.70	9.45	9.45	135.9	251.0		1.57	1.27	0.89	1.08	0.61
29	11.2	41.5	4.80	40	HEB 140	82.21	6.39	0.70	9.45	9.45	135.9	265.0		1.57	1.35	0.96	1.16	0.65
30	11.3	41.5	4.18	40	HEB 140	82.21	6.39	0.70	9.45	9.45	135.9	240.0		1.57	1.26	0.93	1.13	0.64
31	12.1	55.0	4.67	40	HEB 140	82.14	6.47	0.70	9.45	9.45	135.9	265.0		1.57	1.16	0.90	1.08	0.59
32	12.2	55.0	4.52	40	HEB 140	82.14	6.47	0.70	9.45	9.45	135.9	251.0		1.57	1.10	0.86	1.04	0.57
33	12.3	55.0	4.18	40	HEB 140	82.14	6.47	0.70	9.45	9.45	135.9	223.0		1.57	1.00	0.79	0.96	0.53
34	13.1	39.5	4.81	40	IPE 220	98.10	5.41	0.70	12.60	8.27	96.5	269.0		1.57	1.40	0.94	1.12	0.72
35	13.2	39.5	4.52	40	IPE 220	98.06	5.45	0.70	12.60	8.27	96.5	234.0		1.57	1.24	0.85	1.01	0.65
36	13.3	39.5	4.44	40	IPE 220	98.06	5.45	0.70	12.60	8.27	96.5	229.0		1.57	1.22	0.84	1.00	0.64
Loke, 1968																		
37	5	40.7	3.71	40	BS 4 x 3	53.11	2.89		8.00	7.00	89.0	195.0		0.40	1.42	1.29	1.28	0.81
38	6	45.6	3.28	40	BS 4 x 3	53.15	2.85		8.00	7.00	89.0	108.0		0.80	1.01	0.86	1.03	0.56
39	7	39.3	4.20	40	BS 4 x 3	53.11	2.89		8.00	7.00	89.0	88.0		1.50	1.18	0.77	1.06	0.61
40	9	39.5	4.58	40	BS 4 x 3	53.09	2.91		8.00	7.00	125.0	201.0		0.20	1.30	1.73	1.30	0.68
41	10	39.5	4.31	40	BS 4 x 3	53.11	2.89		8.00	7.00	125.0	135.0		0.40	1.08	1.25	1.08	0.51
42	11	42.7	3.25	40	BS 4 x 3	53.19	2.81		8.00	7.00	125.0	88.0		0.80	0.96	1.03	1.11	0.47
43	12	39.5	4.28	40	BS 4 x 3	53.09	2.91		8.00	7.00	125.0	67.5		1.50	0.98	0.79	0.99	0.46
44	2	42.4	4.28	40	BS 4 x 3	53.17	2.83		8.00	7.00	125.0	210.8		0.40	1.64	1.98	1.43	0.95
45	3	42.4	3.91	40	BS 4 x 3	53.17	2.83		8.00	7.00	125.0	129.5		0.80	1.35	1.38	1.43	0.62
46	14	43.0	2.89	40	BS 4 x 1.75	54.49	1.51		8.00	7.00	89.0	116.0		0.40	1.33	1.20	1.13	0.70
47	15	43.0	3.81	40	BS 4 x 1.75	54.49	1.51		8.00	7.00	89.0	108.0		0.80	1.51	1.06	1.09	0.62
48	17	39.5	3.81	40	BS 4 x 3	53.09	2.91		7.00	8.00	89.0	214.0	0.40		1.38	1.04	1.04	0.87
49	18	39.5	3.46	40	BS 4 x 3	53.09	2.91		7.00	8.00	89.0	175.0	0.80		1.41	1.00	1.07	0.87
Roik and Schwalbenhofer, 1989																		
50	V11	37.1	6.35	60.9	HE 120 B	115.30	5.27	0.95	11.02	11.02	118.1	171.5	6.30		1.60	0.66	1.17	0.81
51	V12	37.1	6.35	60.9	HE 120 B	115.30	5.27	0.95	11.02	11.02	118.1	366.0	2.36		1.73	0.87	0.96	0.72
52	V13	37.1	6.79	60.9	HE 120 B	115.30	5.27	0.95	11.02	11.02	118.1	322.6	3.94		2.11	0.92	1.26	0.86

Table A-2 - SRC Beam-Column Database

Col. No.	Spec. No.	Fy (ksi)	f'c (ksi)	Fyr (ksi)	Steel Section	Ac (in^2)	As (in^2)	Ar (in^2)	h1 (in)	h2 (in)	kl (in)	Pexp (k)	ey (in)	ex (in)	Pexp/Ppred by AISC 1999	AISC 2005	Eurocode 4	Plastic
53	V21	49.5	6.79	60.9	HE 160 A	114.56	6.01	0.95	11.02	11.02	118.1	337.9	3.94		1.54	0.79	1.01	0.76
54	V22	49.5	5.37	60.9	HE 160 A	114.56	6.01	0.95	11.02	11.02	118.1	213.6	6.30		1.38	0.74	1.12	0.79
55	V23	49.5	5.37	60.9	HE 160 A	114.56	6.01	0.95	11.02	11.02	118.1	436.8	2.36		1.56	0.95	1.05	0.80
56	V31	37.8	5.90	60.9	HE 200 B	108.46	12.11	0.95	11.02	11.02	118.1	383.8	3.94		1.18	0.75	0.94	0.72
57	V32	37.8	5.90	60.9	HE 200 B	108.46	12.11	0.95	11.02	11.02	118.1	506.5	2.36		1.20	0.81	0.91	0.77
58	V33	37.8	5.70	60.9	HE 200 B	108.46	12.11	0.95	11.02	11.02	118.1	294.1	6.30		1.20	0.75	1.03	0.80
59	V41	49.3	5.70	60.9	HE 180 M	103.05	17.52	0.95	11.02	11.02	118.1	477.3	3.94		0.95	0.72	0.87	0.71
60	V42	49.3	6.12	60.9	HE 180 M	103.05	17.52	0.95	11.02	11.02	118.1	344.6	6.30		0.88	0.65	0.85	0.65
61	V43	49.5	6.12	60.9	HE 180 M	103.05	17.52	0.95	11.02	11.02	118.1	614.4	2.36		0.95	0.73	0.85	0.70
62	V71	45.5	5.66	60.9	HE 180 M	103.05	17.52	0.95	11.02	11.02	118.1	360.4	-6.3/7.87					
63	V72	45.5	6.05	60.9	HE 180 M	103.05	17.52	0.95	11.02	11.02	118.1	441.3	5.5/6.30					
64	V73	45.5	6.05	60.9	HE 180 M	103.05	17.52	0.95	11.02	11.02	118.1	607.5	3.94/0.00					
65	V111	45.3	5.82	60.9	HE 160 A	111.69	6.01	3.82	11.02	11.02	118.1	394.6	3.94		1.50	0.90	0.98	0.74
66	V112	45.3	5.82	60.9	HE 160 A	111.69	6.01	3.82	11.02	11.02	118.1	565.4	2.36		1.65	1.03	1.01	0.84
67	V121	35.1	5.82	60.9	HE 120 B	112.43	5.27	3.82	11.02	11.02	118.1	255.8	6.30		1.65	0.88	1.05	0.78
68	V122	35.1	5.82	60.9	HE 120 B	112.43	5.27	3.82	11.02	11.02	118.1	182.8	7.87		1.38	0.73	0.95	0.68
69	V123	35.1	5.82	60.9	HE 120 B	112.43	5.27	3.82	11.02	11.02	118.1	345.1	3.94		1.63	0.91	0.97	0.73
Roik, Mangerig, and Schwalbenhofer, 1990																		
70	7	39.2	6.09	60.9	HE 200 B	126.69	12.11	0.70	11.81	11.81	118.1	1022.3		1.18	2.03	1.43	1.50	1.25
71	8	39.2	6.09	60.9	HE 200 B	126.69	12.11	0.70	11.81	11.81	118.1	501.6		3.54	1.77	1.07	1.34	1.04
72	9	39.2	6.09	60.9	HE 200 B	126.69	12.11	0.70	11.81	11.81	196.9	824.0		1.18	1.97	1.50	1.55	1.01
73	10	39.2	6.09	60.9	HE 200 B	126.69	12.11	0.70	11.81	11.81	196.9	410.5		3.54	1.65	1.05	1.34	0.85
74	11	39.2	6.09	60.9	HE 200 B	126.69	12.11	0.70	11.81	11.81	315.0	454.6		1.18	1.60	1.61	1.39	0.56
75	12	39.2	6.09	60.9	HE 200 B	126.69	12.11	0.70	11.81	11.81	315.0	223.7		3.54	1.20	0.96	1.19	0.46
76	23	39.2	6.09	60.9	HE 200 B	126.69	12.11	0.70	11.81	11.81	196.9	525.9	3.54		1.66	0.99	1.28	0.84
77	24	39.2	6.09	60.9	HE 200 B	126.69	12.11	0.70	11.81	11.81	196.9	368.0	5.91		1.54	0.87	1.30	0.86
78	25	39.2	6.09	60.9	HE 200 B	126.69	12.11	0.70	11.81	11.81	315.0	377.5	3.54		1.66	1.02	1.38	0.60
79	26	39.2	6.09	60.9	HE 200 B	126.69	12.11	0.70	11.81	11.81	315.0	200.8	5.91		1.11	0.64	1.01	0.47
80	27	39.2	6.09	60.9	HE 200 M	118.50	20.31	0.70	11.81	11.81	196.9	939.7	1.18		1.34	1.01	1.12	0.84

Table A-2 - SRC Beam-Column Database

Col. No.	Spec. No.	Fy (ksi)	f'c (ksi)	Fyr (ksi)	Steel Section	Ac (in^2)	As (in^2)	Ar (in^2)	h1 (in)	h2 (in)	kl (in)	Pexp (k)	ey (in)	ex (in)	Pexp/Ppred by AISC 1999	AISC 2005	Eurocode 4	Plastic
81	28	39.2	6.09	60.9	HE 200 M	118.50	20.31	0.70	11.81	11.81	315.0	519.3	1.18		1.08	0.79	0.89	0.46
82	29	39.2	6.09	60.9	HE 200 M	118.50	20.31	0.70	11.81	11.81	196.9	786.9	1.18		1.12	0.85	0.94	0.70
83	30	39.2	6.09	60.9	HE 200 M	118.50	20.31	0.70	11.81	11.81	315.0	424.9	1.18		0.89	0.64	0.73	0.38
A. Mirza, V. Hyttinen, and E. Hyttinen, 1997																		
84	RHB-1	42.5	3.95	81.93	HE100A	85.95	3.29	0.49	9.46	9.46	157.6	213.8	1.56		1.71	1.21	1.20	0.69
85	RHB-2	42.5	3.95	81.93	HE100A	85.95	3.29	0.49	9.46	9.46	157.6	123.8	2.59		1.28	0.82	0.92	0.54
86	RHB-3	42.5	4.02	81.93	HE100A	85.95	3.29	0.49	9.46	9.46	157.6	128.3	4.16		1.77	1.02	1.40	0.80
87	RHB-4	45.1	3.75	91.93	HE100A	85.95	3.29	0.49	9.46	9.46	157.6	69.2	7.89		1.43	0.80	1.30	0.86
88	RHB-4A	42.5	3.64	81.93	HE100A	85.95	3.29	0.49	9.46	9.46	157.6	34.7	14.61		1.26	0.64	1.23	0.89
89	RHB-5	42.5	4.15	81.93	HE100A	85.95	3.29	0.49	9.46	9.46	157.6	21.4	24.96					
90	RNHB-1	45.1	3.99	91.93	HE100A	85.95	3.29	0.49	9.46	9.46	157.6	208.1	1.94		1.73	1.18	1.21	0.69
91	RNHB-2	45.1	3.99	91.93	HE100A	85.95	3.29	0.49	9.46	9.46	157.6	174.4	2.27		1.56	1.04	1.10	0.63
92	RNHB-3	42.5	3.88	81.93	HE100A	85.95	3.29	0.49	9.46	9.46	157.6	121.5	4.17		1.63	0.95	1.28	0.75
93	RNHB-4	42.5	3.95	81.93	HE100A	85.95	3.29	0.49	9.46	9.46	157.6	79.3	8.24		1.72	0.89	1.52	1.01
94	RNHB-5	42.5	4.08	81.93	HE100A	85.95	3.29	0.49	9.46	9.46	157.6	24.2	24.81					
95	RHNB-1	45.1	3.99	91.93	HE100A	85.95	3.29	0.49	9.46	9.46	157.6	208.6	1.65		1.60	1.13	1.12	0.65
96	RHNB-2	45.1	3.99	91.93	HE100A	85.95	3.29	0.49	9.46	9.46	157.6	162.0	2.34		1.47	0.97	1.04	0.60
97	RHNB-3	45.1	3.75	91.93	HE100A	85.95	3.29	0.49	9.46	9.46	157.6	121.5	3.88		1.49	0.92	1.17	0.69
98	RHNB-4	45.1	3.75	91.93	HE100A	85.95	3.29	0.49	9.46	9.46	157.6	66.6	8.11		1.36	0.75	1.21	0.80
99	RHNB-5	45.1	3.75	91.93	HE100A	85.95	3.29	0.49	9.46	9.46	157.6	22.5	24.43					
K. Roik and C. Diekmann,1989																		
100	11	41.3	6.76	40.0	HE200B	126.89	12.12	0.70	11.82	11.82	118.2	894.4	1.97		1.73	1.12	1.22	1.04
101	12	41.3	6.76	40.0	HE200B	126.89	12.12	0.70	11.82	11.82	118.2	729.0	1.97		1.41	0.92	0.98	0.82
102	13	41.3	6.76	40.0	HE200B	126.89	12.12	0.70	11.82	11.82	118.2	852.5	1.97		1.65	1.07	1.16	0.99
103	14	35.8	6.76	40.0	HE160A	132.99	6.02	0.70	11.82	11.82	118.2	640.1	1.97		2.31	1.12	1.25	0.97
104	15	58.0	6.76	40.0	HE200B	126.89	12.12	0.70	11.82	11.82	118.2	985.1	1.97		1.55	1.09	1.19	1.00
105	16	41.3	6.76	40.0	HE200B	126.89	12.12	0.70	11.82	11.82	118.2	603.5	3.94		1.65	0.98	1.26	0.98

Table A-2 - SRC Beam-Column Database

Col. No.	Spec. No.	Fy (ksi)	f'c (ksi)	Fyr (ksi)	Steel Section	Ac (in^2)	As (in^2)	Ar (in^2)	h1 (in)	h2 (in)	kl (in)	Pexp (k)	ey (in)	ex (in)	Pexp/Ppred by AISC 1999	AISC 2005	Eurocode 4	Plastic
Han et al.,1992																		
106	ALH-E-80	33.6	3.0	40.6	H-100x75x3.2x4.5	37.74	1.50	0.44	6.30	6.30	19.69	183.4	0.79		2.54	1.50	1.50	1.43
107	ALH-E-40	33.6	3.0	40.6	H-100x75x3.2x4.5	37.74	1.50	0.44	6.30	6.30	19.69	203.7	0.79		2.82	1.66	1.67	1.59
108	ARH-E-80	44.8	3.1	40.6	H-100x100x6x8	35.84	3.39	0.44	6.30	6.30	19.69	269.4	0.79		1.96	1.40	1.49	1.40
109	ARH-E-40	44.8	3.1	40.6	H-100x100x6x8	35.84	3.39	0.44	6.30	6.30	19.69	285.7	0.79		2.08	1.49	1.58	1.48
Han and Kim,1995																		
110	AH2-E2-80	45.8	3.1	40.6	H-100x100x6x8	35.84	3.39	0.44	6.30	6.30	27.56	269.4	0.79		2.01	1.43	1.52	1.41
111	AH2-E4-80	45.8	3.1	40.6	H-100x100x6x8	35.84	3.39	0.44	6.30	6.30	27.56	185.2	1.57		1.85	1.29	1.44	1.29
112	AH2-E8-80	45.8	3.1	40.6	H-100x100x6x8	35.84	3.39	0.44	6.30	6.30	27.56	118.4	3.15		1.63	1.11	1.28	1.11
113	BH2-E2-80	45.8	3.1	40.6	H-100x100x6x8	35.84	3.39	0.44	6.30	6.30	47.24	218.3	0.79		1.76	1.24	1.40	1.21
114	BH2-E4-80	45.8	3.1	40.6	H-100x100x6x8	35.84	3.39	0.44	6.30	6.30	47.24	168.4	1.57		1.85	1.27	1.52	1.28
115	BH2-E8-80	45.8	3.1	40.6	H-100x100x6x8	35.84	3.39	0.44	6.30	6.30	47.24	112.2	3.15		1.59	1.07	1.28	1.05
116	CH1-E2-80	45.8	3.1	40.6	H-100x75x3.2x4.5	37.74	1.50	0.44	6.30	6.30	66.93	197.1	0.79		2.60	1.59	1.63	1.37
117	CH1-E4-80	45.8	3.1	40.6	H-100x75x3.2x4.5	37.74	1.50	0.44	6.30	6.30	66.93	126.5	1.57		2.19	1.25	1.37	1.08
118	CH1-E8-80	45.8	3.1	40.6	H-100x75x3.2x4.5	37.74	1.50	0.44	6.30	6.30	66.93	77.6	3.15		1.96	1.04	1.42	1.04
119	CH2-E2-80	45.8	3.1	40.6	H-100x100x6x8	35.84	3.39	0.44	6.30	6.30	66.93	221.1	0.79		2.09	1.43	1.85	1.46
120	CH2-E4-80	45.8	3.1	40.6	H-100x100x6x8	35.84	3.39	0.44	6.30	6.30	66.93	153.9	1.57		1.59	1.08	1.24	1.01
121	CH2-E8-80	45.8	3.1	40.6	H-100x100x6x8	35.84	3.39	0.44	6.30	6.30	66.93	107.1	3.15		1.59	1.05	1.30	1.00
122	DH2-E2-80	45.8	3.1	40.6	H-100x100x6x8	35.84	3.39	0.44	6.30	6.30	86.61	190.5	0.79		2.07	1.39	2.08	1.50
123	DH2-E4-80	45.8	3.1	40.6	H-100x100x6x8	35.84	3.39	0.44	6.30	6.30	86.61	137.6	1.57		1.53	1.02	1.20	0.90
124	DH2-E8-80	45.8	3.1	40.6	H-100x100x6x8	35.84	3.39	0.44	6.30	6.30	86.61	107.8	3.15		1.69	1.10	1.42	1.01
Biaxial Bending																		
Virdi and Dowling, 1972																		
1	A	46.5	7.38	45.6	UC152x23	94.62	4.62	0.763	10.00	10.00	72.0	282.1	2.50	1.45				
2	B	46.5	7.06	45.6	UC152x23	94.62	4.62	0.763	10.00	10.00	72.0	145.7	5.00	2.90				
3	C	46.5	7.38	45.6	UC152x23	94.62	4.62	0.763	10.00	10.00	72.0	106.3	7.50	4.35				
4	D	46.5	7.82	45.6	UC152x23	94.62	4.62	0.763	10.00	10.00	144.1	208.4	2.50	1.45				
5	E	46.5	7.38	45.6	UC152x23	94.62	4.62	0.763	10.00	10.00	144.1	128.8	5.00	2.90				

Table A-2 - SRC Beam-Column Database

Col. No.	Spec. No.	Fy (ksi)	f'c (ksi)	Fyr (ksi)	Steel Section	Ac (in^2)	As (in^2)	Ar (in^2)	h1 (in)	h2 (in)	kl (in)	Pexp (k)	ey (in)	ex (in)	Pexp/Ppred by AISC 1999	AISC 2005	Eurocode 4	Plastic
6	F	46.5	7.82	45.6	UC152x23	94.62	4.62	0.763	10.00	10.00	144.1	94.0	7.50	4.35				
7	G	46.5	6.77	45.6	UC152x23	94.62	4.62	0.763	10.00	10.00	288.2	151.3	2.50	1.45				
8	H	46.5	7.38	45.6	UC152x23	94.62	4.62	0.763	10.00	10.00	288.2	79.6	5.00	2.90				
9	I	46.5	8.04	45.6	UC152x23	94.62	4.62	0.763	10.00	10.00	288.2	66.1	7.50	4.35				
Johnson and May, 1978																		
10	BC1	40.2	3.39		152x89 RSJ17.1	39.98	3.38	0	5.51	7.87	102.4	166.8	1.14	0.24				
11	BC2	40.2	3.24		152x89 RSJ17.1	39.98	3.38	0	5.51	7.87	102.4	187.5	0.97	0.17				
12	BC3	40.2	3.53		152x89 RSJ17.1	39.98	3.38	0	5.51	7.87	102.4	164.1	1.22	0.27				

Table A-3 - CCFT Column Database

Col. No.	Spec. No.	Fy (ksi)	f'c (ksi)	D (in)	t (in)	Ac (in^2)	As (in^2)	kl (in)	Pexp (k)	Pexp/Ppred by AISC 1999	AISC 2005	Eurocode 4	Plastic
Gardner and Jacobson, 1967													
1	22	52.7	5.93	3.01	0.066	6.50	0.61	6.0	97.7	1.51	1.42	1.09	1.38
2	23	52.7	3.76	3.01	0.067	6.49	0.62	6.0	83.7	1.57	1.50	1.10	1.46
3	19	52.7	3.62	3.01	0.067	6.49	0.62	6.0	80.0	1.53	1.46	1.06	1.43
4	3	87.8	4.95	4.00	0.121	11.11	1.48	8.0	250.0	1.42	1.38	0.99	1.35
5	4	87.8	4.52	4.00	0.121	11.12	1.47	8.0	240.0	1.40	1.36	0.97	1.34
6	8	65.5	4.99	4.76	0.160	15.46	2.31	9.5	270.0	1.25	1.21	0.87	1.18
7	9	65.5	4.29	4.76	0.161	15.45	2.32	9.5	270.0	1.30	1.26	0.90	1.24
8	10	65.5	3.76	4.76	0.161	15.46	2.32	9.5	250.0	1.25	1.21	0.85	1.19
9	13	60.2	3.03	6.01	0.125	26.06	2.30	12.0	270.0	1.32	1.27	0.90	1.24
10	14	60.2	3.35	6.01	0.124	26.06	2.29	12.0	270.0	1.28	1.23	0.88	1.20
11	15	91.9	6.09	6.01	0.194	24.79	3.55	12.0	654.0	1.45	1.40	1.02	1.37
12	16	91.9	6.30	6.01	0.193	24.80	3.53	12.0	655.0	1.44	1.39	1.01	1.36
Gardner, 1968													
13	1a	43.2	2.60	6.64	0.104	32.49	2.14	12.0	298.0	1.82	1.73	1.26	1.69
14	2a	43.2	4.95	6.64	0.104	32.49	2.14	12.0	274.0	1.20	1.12	0.88	1.08
15	2a	46.0	5.30	6.66	0.103	32.72	2.12	12.0	294.0	1.20	1.12	0.88	1.08
16	4a	46.0	4.87	6.66	0.103	32.72	2.12	12.0	299.0	1.29	1.20	0.94	1.16
17	5a	32.1	3.86	6.62	0.142	31.53	2.89	12.0	350.0	1.79	1.68	1.28	1.63
18	6a	32.1	4.75	6.62	0.142	31.53	2.89	12.0	322.0	1.47	1.37	1.07	1.33
19	6b	32.1	4.75	6.62	0.142	31.53	2.89	12.0	329.0	1.50	1.40	1.09	1.36
20	7a	37.8	4.77	6.64	0.197	30.64	3.99	12.0	442.0	1.61	1.53	1.13	1.49
21	7b	37.8	4.77	6.64	0.197	30.64	3.99	12.0	443.0	1.61	1.53	1.14	1.49
22	8a	37.8	3.98	6.64	0.197	30.64	3.99	12.0	446.0	1.76	1.68	1.22	1.64
23	8b	37.8	3.98	6.64	0.197	30.64	3.99	12.0	446.0	1.76	1.68	1.22	1.64
Chapman and Neogi, 1966													
24	A1	51.5	5.52	14.00	0.440	135.19	18.74	74.0	2576.0	1.36	1.57	1.36	1.52
25	A4	51.5	4.76	14.00	0.440	135.19	18.74	74.0	2408.0	1.34	1.56	1.34	1.51

Table A-3 - CCFT Column Database

Col. No.	Spec. No.	Fy (ksi)	f'c (ksi)	D (in)	t (in)	Ac (in^2)	As (in^2)	kl (in)	Pexp (k)	Pexp/Ppred by AISC 1999	AISC 2005	Eurocode 4	Plastic
26	A5	40.1	3.04	14.00	0.186	145.87	8.07	74.0	790.7	0.94	1.08	0.94	1.04
27	A6	51.5	3.40	14.00	0.316	140.35	13.58	82.0	1671.0	1.55	1.49	1.30	1.44
28	B1	53.8	9.60	5.01	0.064	18.64	0.99	28.0	289.0	1.45	1.34	1.22	1.26
29	B1X	47.7	9.60	5.01	0.064	18.64	0.99	28.0	289.0	1.49	1.37	1.25	1.29
30	B2	53.8	9.60	5.00	0.116	17.86	1.78	28.0	293.5	1.38	1.38	1.15	1.32
31	B2X	47.7	9.60	5.00	0.116	17.86	1.78	28.0	293.5	1.31	1.22	1.10	1.16
32	DF1	38.5	4.66	5.51	0.381	17.71	6.14	16.0	663.0	2.17	2.12	1.59	2.08
33	DF1X	39.0	4.66	5.51	0.381	17.71	6.14	16.0	663.0	2.15	2.10	1.58	2.06
34	DF2	41.9	4.74	5.53	0.194	20.77	3.25	16.0	410.0	1.87	1.79	1.37	1.75
35	DF2X	43.2	4.74	5.53	0.194	20.77	3.25	16.0	410.0	1.84	1.76	1.35	1.72
36	SC1	43.2	4.56	6.62	0.178	30.82	3.60	32.0	451.0	1.66	1.59	1.33	1.52
37	SC2	43.2	6.26	6.63	0.178	30.92	3.61	32.0	502.0	1.59	1.50	1.29	1.44
38	SC3	43.2	6.26	6.62	0.178	30.82	3.60	32.0	475.0	1.51	1.43	1.22	1.37
39	SC4	43.2	3.34	6.63	0.176	30.96	3.57	32.0	392.0	1.64	1.58	1.30	1.52
Sakino and Hayashi, 1991													
40	L-20-1	41.1	3.21	7.01	0.354	31.16	7.41	9.9	657.0	1.69	1.65	1.11	1.62
41	L-20-2	41.1	3.21	7.01	0.354	31.16	7.41	9.9	641.5	1.65	1.61	1.08	1.59
42	H-20-1	41.1	6.59	7.01	0.354	31.16	7.41	9.9	723.1	1.51	1.45	1.04	1.42
43	H-20-2	41.1	6.59	7.01	0.354	31.16	7.41	9.9	714.3	1.49	1.43	1.03	1.40
44	L-32-1	36.0	3.21	7.05	0.217	34.36	4.65	9.9	407.9	1.56	1.50	1.04	1.47
45	L-32-2	36.0	3.47	7.05	0.217	34.36	4.65	9.9	407.9	1.52	1.45	1.02	1.42
46	H-32-1	36.0	6.33	7.05	0.217	34.36	4.65	9.9	458.6	1.30	1.23	0.92	1.19
47	H-32-2	36.0	6.33	7.05	0.217	34.36	4.65	9.9	456.4	1.30	1.22	0.92	1.19
48	L-58-1	38.6	3.47	6.85	0.118	34.36	2.50	9.9	295.4	1.50	1.41	1.05	1.37
49	L-58-2	38.6	3.47	6.85	0.118	34.36	2.50	9.9	293.2	1.48	1.40	1.04	1.36
50	H-58-1	38.6	6.63	6.85	0.118	34.36	2.50	9.9	361.6	1.25	1.16	0.93	1.12
51	H-58-2	38.6	6.63	6.85	0.118	34.36	2.50	9.9	377.0	1.30	1.21	0.97	1.16

Table A-3 - CCFT Column Database

Col. No.	Spec. No.	Fy (ksi)	f'c (ksi)	D (in)	t (in)	Ac (in^2)	As (in^2)	kl (in)	Pexp (k)	Pexp/Ppred by AISC 1999	Pexp/Ppred by AISC 2005	Pexp/Ppred by Eurocode 4	Pexp/Ppred by Plastic
Luksha and Nesterovich, 1991													
52	SB-1	56.8	4.81	6.26	0.200	26.98	3.80	18.8	501.3	1.55	1.49	1.14	1.45
53	SB-2	42.3	4.18	24.80	0.276	461.94	21.24	74.4	3742.9	1.48	1.38	1.15	1.32
54	SB-6	50.7	4.06	24.80	0.300	460.11	23.06	74.4	4046.4	1.48	1.38	1.13	1.33
55	SB-7	50.8	4.00	24.80	0.332	457.63	25.55	74.4	4181.3	1.47	1.39	1.12	1.34
56	SB-3	46.9	4.45	24.80	0.402	452.36	30.81	74.4	4608.4	1.47	1.38	1.11	1.33
57	SB-4	50.3	5.34	24.80	0.457	448.24	34.93	74.4	5485.1	1.46	1.37	1.11	1.32
58	SB-8	45.2	1.74	28.35	0.327	602.32	28.76	85.0	3372.0	1.55	1.48	1.12	1.44
59	SB-5	48.0	5.22	32.28	0.352	783.29	35.27	96.9	7553.3	1.47	1.37	1.15	1.31
60	SB-9	48.7	1.96	40.16	0.380	1219.12	47.43	120.5	6744.0	1.56	1.48	1.16	1.43
61	SB-10	53.5	3.35	40.16	0.522	1201.59	64.96	120.5	10340.8	1.51	1.43	1.13	1.38
Furlong, 1967													
62		60.0	4.20	4.50	0.125	14.19	1.72	36.0	160.0	1.09	1.06	1.01	1.02
63		60.0	4.20	4.50	0.125	14.19	1.72	36.0	170.0	1.16	1.12	1.07	1.09
64		42.0	5.10	5.00	0.095	18.17	1.46	36.0	141.0	1.04	0.98	0.91	0.94
65		42.0	5.10	5.00	0.095	18.17	1.46	36.0	140.0	1.03	0.97	0.90	0.93
66		42.0	5.10	5.00	0.095	18.17	1.46	36.0	148.0	1.09	1.03	0.95	0.98
67		48.0	3.05	6.00	0.061	27.14	1.14	36.0	153.4	1.26	1.18	1.06	1.13
68		48.0	3.75	6.00	0.061	27.14	1.14	36.0	162.2	1.18	1.10	1.00	1.05
69		48.0	3.75	6.00	0.061	27.14	1.14	36.0	164.8	1.20	1.12	1.01	1.07
Gardner, 1968													
70	1	43.2	2.60	6.65	0.104	32.49	2.14	78.0	185.0	1.22	1.17	1.13	1.13
71	2	43.2	4.95	6.65	0.104	32.49	2.14	78.0	206.0	0.99	0.94	0.89	0.89
72	3	46.0	5.30	6.65	0.103	32.72	2.12	78.0	170.0	0.77	0.73	0.69	0.69
73	4	46.0	4.87	6.65	0.103	32.72	2.12	78.0	155.0	0.73	0.70	0.66	0.66
74	5	32.1	3.86	6.62	0.142	31.53	2.89	90.0	213.0	1.20	1.14	1.09	1.09
75	6	32.1	4.75	6.62	0.142	31.53	2.89	90.0	236.0	1.19	1.13	1.07	1.07

Table A-3 - CCFT Column Database

Col. No.	Spec. No.	Fy (ksi)	f'c (ksi)	D (in)	t (in)	Ac (in^2)	As (in^2)	kl (in)	Pexp (k)	Pexp/Ppred by AISC 1999	AISC 2005	Eurocode 4	Plastic
76	7	37.8	4.77	6.64	0.197	30.64	3.99	90.0	254.0	1.03	0.99	0.94	0.94
77	8	37.8	3.98	6.64	0.197	30.64	3.99	90.0	262.0	1.14	1.10	1.05	1.05
Gardner and Jacobson, 1967													
78	1	87.8	4.95	4.00	0.121	11.09	1.47	60.0	184.0	1.32	1.29	1.24	1.24
79	2	87.8	4.52	4.01	0.121	11.08	1.49	60.0	180.0	1.31	1.29	1.24	1.24
80	5	65.5	4.99	4.75	0.161	15.40	2.32	41.3	260.0	1.28	1.24	1.20	1.20
81	6	65.5	4.29	4.76	0.161	15.47	2.33	41.3	245.6	1.25	1.22	1.19	1.18
82	7	65.5	3.76	4.76	0.161	15.47	2.33	41.3	213.5	1.13	1.10	1.07	1.07
83	11	60.2	3.03	6.01	0.124	25.99	2.29	66.0	211.0	1.13	1.09	1.05	1.05
84	12	60.2	3.03	6.01	0.124	26.08	2.29	66.0	198.0	1.05	1.02	0.99	0.99
85	18	52.7	3.62	3.01	0.067	6.50	0.62	60.0	55.0	1.37	1.34	1.31	1.31
86	20	52.7	5.93	3.01	0.067	6.49	0.63	24.0	92.5	1.48	1.40	1.33	1.34
87	21	52.7	3.76	3.01	0.067	6.49	0.63	24.0	74.3	1.44	1.38	1.31	1.34
Salani and Sims, 1964													
88	30F	76.0	4.04	2.00	0.065	2.75	0.40	42.0	27.1	1.03	1.01	1.03	1.03
89	42F	76.0	3.95	3.00	0.065	6.47	0.60	42.0	72.0	1.27	1.24	1.19	1.19
90	49F	76.0	4.04	1.00	0.035	0.68	0.11	42.0	3.5	1.52	1.51	1.58	1.58
91	50F	76.0	4.04	1.00	0.035	0.68	0.11	42.0	3.5	1.50	1.49	1.55	1.55
92	51F	76.0	4.04	1.50	0.109	1.29	0.48	42.0	25.4	1.38	1.39	1.50	1.50
93	52F	76.0	4.04	1.50	0.109	1.29	0.48	42.0	24.0	1.31	1.31	1.42	1.42
94	71F	76.0	4.04	2.75	0.049	5.52	0.42	42.0	51.9	1.26	1.23	1.17	1.17
Knolwes and Park, 1969													
95	1	58.0	5.81	3.50	0.230	7.26	2.36	68.0	138.2	1.07	1.06	1.02	1.02
96	2	58.0	5.75	3.50	0.230	7.26	2.36	56.0	160.0	1.13	1.11	1.06	1.06
97	3	58.0	5.65	3.50	0.230	7.26	2.36	44.0	160.8	1.06	1.04	1.00	1.00
98	4	58.0	6.06	3.50	0.230	7.26	2.36	32.0	206.5	1.26	1.24	1.21	1.20
99	5	58.0	5.93	3.50	0.230	7.26	2.36	20.0	223.0	1.32	1.29	1.13	1.25

Table A-3 - CCFT Column Database

Col. No.	Spec. No.	Fy (ksi)	f'c (ksi)	D (in)	t (in)	Ac (in^2)	As (in^2)	kl (in)	Pexp (k)	Pexp/Ppred by AISC 1999	AISC 2005	Eurocode 4	Plastic
100	6	70.0	6.00	3.25	0.055	7.74	0.55	68.0	50.5	0.95	0.94	0.90	0.90
101	7	70.0	5.37	3.25	0.055	7.74	0.55	56.0	66.2	1.16	1.13	1.06	1.06
102	8	70.0	5.93	3.25	0.055	7.74	0.55	44.0	80.0	1.21	1.17	1.09	1.09
103	9	70.0	5.93	3.25	0.055	7.74	0.55	32.0	90.0	1.27	1.21	1.15	1.14
104	10	70.0	5.93	3.25	0.055	7.74	0.55	20.0	110.0	1.47	1.39	1.27	1.33
105	11	70.0	5.93	3.25	0.055	7.74	0.55	10.0	119.2	1.55	1.47	1.18	1.41
Masuo, Adachi, Kawabata, Kobayashi, and Konishi, 1991													
106	1A2	73.2	8.11	7.51	0.236	38.87	5.40	45.3	688.5	1.08	1.03	0.94	0.99
107	1A4	73.2	8.11	7.51	0.236	38.87	5.40	90.6	587.1	1.04	0.99	0.93	0.93
108	1A6	73.2	8.11	7.51	0.236	38.87	5.40	135.8	463.0	1.01	0.94	0.89	0.89
109	1G2	73.2	7.01	7.51	0.236	38.87	5.40	45.3	707.7	1.17	1.12	1.02	1.08
110	1G6	73.2	7.01	10.53	0.236	38.87	5.40	135.8	479.5	1.05	0.81	0.78	0.78
111	2A2	66.8	8.11	10.53	0.276	78.17	8.88	63.0	1164.7	1.11	1.01	0.92	0.97
112	2A4	66.8	8.11	10.53	0.276	78.17	8.88	126.0	1019.2	1.22	0.99	0.93	0.93
113	2A6	66.8	8.11	10.53	0.276	78.17	8.88	189.0	815.0	1.44	0.95	0.89	0.89
114	2G2	66.8	7.01	10.53	0.276	78.17	8.88	63.0	1166.2	1.18	1.08	0.98	1.04
115	2G6	66.8	7.01	10.53	0.276	78.17	8.88	189.0	877.4	1.50	1.08	1.02	1.02
Janss and Guiaux, 1970													
116	1	43.7	4.55	8.59	0.238	51.74	6.25	171.9	385.7	1.06	1.02	0.98	0.98
117	2	43.7	4.88	8.59	0.254	51.33	6.65	129.3	463.9	1.07	1.03	0.98	0.98
118	3	43.7	4.29	8.59	0.248	51.48	6.50	86.8	542.2	1.23	1.18	1.14	1.13
119	4	43.7	4.29	8.59	0.256	51.28	6.70	37.1	619.3	1.31	1.25	1.02	1.21
120	5	43.7	4.55	8.60	0.251	51.53	6.58	37.1	617.1	1.28	1.22	1.00	1.18
121	6	43.7	4.55	8.63	0.238	52.24	6.28	37.1	630.3	1.34	1.28	1.05	1.23
122	8.1	40.7	4.55	3.74	0.150	9.30	1.69	170.0	45.4	1.48	1.47	1.55	1.55
123	8.2	40.7	4.88	3.75	0.146	9.40	1.65	170.1	34.8	1.13	1.12	1.18	1.18
124	8.3	40.7	4.88	3.76	0.143	9.48	1.62	170.1	31.5	1.03	1.02	1.07	1.07
125	9.1	40.7	4.88	3.75	0.149	9.36	1.68	112.0	62.8	1.01	0.99	1.05	1.05

Table A-3 - CCFT Column Database

Col. No.	Spec. No.	Fy (ksi)	f'c (ksi)	D (in)	t (in)	Ac (in^2)	As (in^2)	kl (in)	Pexp (k)	Pexp/Ppred by AISC 1999	AISC 2005	Eurocode 4	Plastic
126	9.2	40.7	4.29	3.75	0.146	9.40	1.65	112.0	63.3	1.06	1.04	1.10	1.10
127	9.3	40.7	4.29	3.76	0.151	9.40	1.71	112.0	65.5	1.07	1.05	1.11	1.11
128	10.1	40.7	4.29	3.76	0.147	9.44	1.66	76.5	81.5	1.02	0.99	0.95	0.95
129	10.2	40.7	4.88	3.75	0.149	9.36	1.68	76.5	91.5	1.10	1.07	1.03	1.03
130	10.3	40.7	4.88	3.75	0.149	9.36	1.68	76.5	91.5	1.10	1.07	1.03	1.03
131	11.1	40.7	4.88	3.76	0.148	9.43	1.68	57.8	99.8	1.08	1.04	0.99	0.99
132	11.2	40.7	4.88	3.75	0.147	9.38	1.66	57.8	99.2	1.08	1.04	0.99	0.99
133	11.3	40.7	4.88	3.74	0.146	9.34	1.64	57.8	111.3	1.22	1.18	1.12	1.12
134	12.1	40.7	4.88	3.76	0.146	9.45	1.65	39.3	117.9	1.18	1.14	1.11	1.10
135	12.2	40.7	4.88	3.75	0.140	9.46	1.59	39.1	113.9	1.17	1.13	1.09	1.09
136	12.3	40.7	4.88	3.75	0.145	9.41	1.64	39.2	119.9	1.21	1.17	1.13	1.12
137	13.1	40.7	4.88	3.75	0.147	9.38	1.66	19.8	143.3	1.37	1.31	1.11	1.27
138	13.2	40.7	4.88	3.76	0.148	9.43	1.68	19.8	142.2	1.35	1.29	1.09	1.25
139	13.3	40.7	4.88	3.76	0.147	9.44	1.66	19.9	149.9	1.43	1.37	1.16	1.32
Janss, 1974													
140	1	51.6	5.03	15.98	0.197	190.90	9.76	62.6	1697.1	1.30	1.22	1.04	1.16
141	2	51.6	5.54	15.98	0.197	190.90	9.76	63.8	1697.1	1.23	1.14	0.98	1.09
142	3	51.6	5.03	15.98	0.197	190.90	9.76	47.0	1730.1	1.32	1.23	1.02	1.18
143	4	51.6	5.22	15.98	0.197	190.90	9.76	47.6	1675.0	1.25	1.16	0.97	1.12
144	7	46.6	4.01	14.02	0.319	140.56	13.72	59.2	1586.9	1.44	1.37	1.12	1.32
145	8	46.6	4.01	14.02	0.323	140.40	13.89	59.0	1531.8	1.38	1.31	1.07	1.27
146	9	46.6	4.01	14.02	0.305	141.14	13.14	58.9	1615.5	1.49	1.42	1.17	1.37
147	10	90.8	4.01	10.83	0.354	80.41	11.66	56.3	1840.3	1.42	1.39	1.23	1.36
148	11	90.8	4.01	10.83	0.354	80.41	11.66	56.1	1829.3	1.41	1.38	1.22	1.35
149	12	90.8	4.01	10.83	0.354	80.41	11.66	56.7	1884.4	1.46	1.42	1.26	1.39
150	13	90.8	4.01	10.83	0.354	80.41	11.66	56.7	1807.3	1.40	1.36	1.21	1.33
151	14	90.8	4.01	10.83	0.354	80.41	11.66	58.6	1873.4	1.45	1.42	1.27	1.38
152	15	90.8	4.01	10.83	0.354	80.41	11.66	58.7	1851.4	1.43	1.40	1.25	1.37

Table A-3 - CCFT Column Database

Col.	Spec.	Fy	f'c	D	t	Ac	As	kl	Pexp	Pexp/Ppred by			
No.	No.	(ksi)	(ksi)	(in)	(in)	(in^2)	(in^2)	(in)	(k)	AISC 1999	AISC 2005	Eurocode 4	Plastic
Yoshoika et al., 1995													
153	CC4-A-2	41.1	3.68	5.87	0.117	24.99	2.11	17.6	211.6	1.29	1.22	0.89	1.19
154	CC4-A-4-1	41.1	5.86	5.86	0.117	24.89	2.10	17.6	239.1	1.14	1.06	0.82	1.03
155	CC4-A-4-2	41.1	5.86	5.87	0.117	24.99	2.11	17.6	242.7	1.15	1.08	0.83	1.04
156	CC4-A-8	41.1	11.14	5.88	0.117	25.06	2.11	17.6	400.2	1.24	1.14	0.95	1.09
157	CC4-C-2	41.1	3.68	11.83	0.117	105.64	4.29	35.5	535.4	1.06	0.99	0.82	0.95
158	CC4-C-4-1	41.1	5.95	11.82	0.117	105.43	4.28	35.5	736.6	1.05	0.96	0.83	0.92
159	CC4-C-4-2	41.1	5.95	11.82	0.117	105.43	4.28	35.5	708.4	1.01	0.93	0.80	0.88
160	CC4-C-8	41.1	11.14	11.83	0.117	105.64	4.29	35.5	1245.0	1.07	0.97	0.87	0.92
161	CC4-D-2	41.1	3.68	17.72	0.117	240.07	6.44	53.1	992.2	0.98	0.91	0.78	0.86
162	CC4-D-4-1	41.1	5.95	17.71	0.117	239.86	6.44	53.1	1543.9	1.05	0.96	0.85	0.91
163	CC4-D-4-2	41.1	5.95	17.72	0.117	240.07	6.44	53.1	1569.9	1.07	0.98	0.87	0.93
164	CC4-D-8	41.1	12.32	17.71	0.117	239.97	6.44	53.1	2621.7	0.95	0.86	0.79	0.81
165	CC6-A-2	83.9	3.68	4.80	0.179	15.50	2.59	14.4	339.2	1.28	1.25	0.84	1.24
166	CC6-A-4-1	83.9	5.86	4.80	0.179	15.47	2.59	14.4	372.5	1.27	1.23	0.86	1.21
167	CC6-A-4-2	83.9	5.86	4.80	0.179	15.47	2.59	14.4	373.8	1.27	1.23	0.86	1.21
168	CC6-A-8	83.9	11.14	4.78	0.179	15.39	2.59	14.4	471.9	1.30	1.25	0.92	1.22
169	CC6-C-2	83.9	3.68	9.39	0.179	64.07	5.17	28.2	682.1	1.08	1.04	0.72	1.02
170	CC6-C-4-1	83.9	5.86	9.38	0.179	63.91	5.17	28.1	805.3	1.07	1.02	0.74	1.00
171	CC6-C-4-2	83.9	5.86	9.37	0.179	63.85	5.16	28.1	819.7	1.09	1.04	0.76	1.02
172	CC6-C-8	83.9	11.14	9.36	0.179	63.68	5.16	28.1	1253.6	1.21	1.14	0.89	1.10
173	CC6-D-2	83.9	3.68	14.20	0.179	150.43	7.87	42.6	1266.0	1.13	1.08	0.85	1.04
174	CC6-D-4-1	83.9	5.95	14.20	0.179	150.43	7.87	42.6	1631.6	1.16	1.09	0.89	1.05
175	CC6-D-4-2	83.9	5.95	14.18	0.179	150.08	7.86	42.5	1583.4	1.13	1.06	0.87	1.02
176	CC6-D-8	83.9	12.32	14.19	0.179	150.25	7.87	42.6	2585.7	1.17	1.08	0.93	1.03
177	CC8-A-2	121.0	3.68	4.25	0.255	11.00	3.20	12.8	511.3	1.22	1.21	0.83	1.20
178	CC8-A-4-1	121.0	5.86	4.27	0.255	11.12	3.21	12.8	549.7	1.24	1.22	0.85	1.21
179	CC8-A-4-2	121.0	5.86	4.26	0.255	11.02	3.20	12.8	539.8	1.22	1.21	0.84	1.19
180	CC8-A-8	121.0	11.14	4.26	0.255	11.07	3.21	12.8	609.6	1.24	1.21	0.87	1.19
181	CC8-C-2	121.0	3.68	8.74	0.255	53.26	6.79	26.2	1115.7	1.13	1.11	0.75	1.10

Table A-3 - CCFT Column Database

Col. No.	Spec. No.	Fy (ksi)	f'c (ksi)	D (in)	t (in)	Ac (in^2)	As (in^2)	kl (in)	Pexp (k)	Pexp/Ppred by AISC 1999	AISC 2005	Eurocode 4	Plastic
182	CC8-C-4-1	121.0	5.86	8.75	0.255	53.31	6.80	26.2	1267.1	1.17	1.14	0.79	1.12
183	CC8-C-4-2	121.0	5.86	8.74	0.255	53.16	6.79	26.2	1284.3	1.19	1.15	0.80	1.13
184	CC8-C-8	121.0	11.14	8.76	0.255	53.41	6.80	26.3	1641.5	1.24	1.19	0.87	1.16
185	CC8-D-2	121.0	3.68	13.26	0.255	127.76	10.41	39.8	1904.7	1.16	1.13	0.87	1.10
186	CC8-D-4-1	121.0	5.95	13.25	0.255	127.45	10.40	39.7	2172.9	1.16	1.11	0.88	1.08
187	CC8-D-4-2	121.0	5.95	13.26	0.255	127.61	10.40	39.8	2210.4	1.17	1.13	0.89	1.10
188	CC8-D-8	121.0	12.32	13.25	0.255	127.53	10.40	39.8	3096.2	1.21	1.14	0.94	1.09
Salani and Sims, 1964													
189	16F	76.0	3.43	1.00	0.035	0.68	0.11	60.0	3.9	2.06	2.06	2.06	2.06
190	17F	76.0	3.43	1.00	0.035	0.68	0.11	60.0	3.3	1.81	1.81	1.77	1.77
191	18F	76.0	3.43	1.00	0.035	0.68	0.11	60.0	3.2	1.78	1.79	1.70	1.70
192	22F	76.0	2.60	1.50	0.109	1.29	0.48	60.0	24.2	1.77	1.80	1.80	1.80
193	23F	76.0	2.60	1.50	0.109	1.29	0.48	60.0	27.2	2.05	2.08	2.05	2.05
194	24F	76.0	2.60	1.50	0.109	1.29	0.48	60.0	24.0	1.86	1.89	1.84	1.84
195	28F	76.0	3.09	2.00	0.065	2.75	0.40	60.0	25.9	1.56	1.56	1.75	1.75
196	29F	76.0	3.09	2.00	0.065	2.75	0.40	60.0	25.7	1.55	1.54	1.74	1.74
197	40F	76.0	3.01	3.00	0.065	6.47	0.60	60.0	50.9	1.16	1.14	1.15	1.15
198	41F	76.0	3.01	3.00	0.065	6.47	0.60	60.0	55.1	1.25	1.24	1.25	1.25
Lin, 1988													
199	D1	36.0	3.00	5.91	0.028	26.88	0.51	18.9	121.0	1.40	1.28	1.13	1.22
200	D2	36.0	3.00	5.91	0.028	26.88	0.51	31.5	115.5	1.35	1.24	1.13	1.18
201	D4	36.0	3.00	5.91	0.055	26.38	1.01	31.5	156.7	1.54	1.43	1.27	1.36
202	D6	36.0	3.00	5.91	0.083	25.88	1.51	31.5	177.0	1.49	1.41	1.21	1.34
203	E1	36.0	5.00	5.91	0.028	26.88	0.51	18.9	167.3	1.27	1.16	1.04	1.10
204	E6	36.0	5.00	5.91	0.083	25.88	1.51	31.5	241.3	1.50	1.39	1.24	1.32
Kenny, Bruce, and Bjorhovde, 1994													
205		98.9	5.57	5.500	0.363	17.90	5.86	36.0	685.0	1.09	1.07	1.04	1.05

Table A-3 - CCFT Column Database

Col. No.	Spec. No.	Fy (ksi)	f'c (ksi)	D (in)	t (in)	Ac (in^2)	As (in^2)	kl (in)	Pexp (k)	Pexp/Ppred by AISC 1999	AISC 2005	Eurocode 4	Plastic
206		98.9	5.57	5.500	0.363	17.90	5.86	36.0	584.0	1.09	1.22	1.03	1.02
207		98.9	5.57	5.500	0.363	17.90	5.86	120.0	450.0	1.47	1.66	1.49	1.49
208		86.1	5.57	7.000	0.502	28.24	10.25	36.0	1181.0	1.41	1.57	1.27	1.32
209		86.1	5.57	7.000	0.502	28.24	10.25	36.0	1242.0	1.48	1.66	1.34	1.39
210		86.1	5.57	7.000	0.502	28.24	10.25	120.0	969.0	1.57	1.77	1.48	1.48
O'Shear and Bridge, 1997													
211	S30CS	52.8	16.46	6.501	0.118	30.82	2.370	22.8	601.4	1.29	1.34	1.26	1.33
212	S20CS	39.4	16.46	7.486	0.079	42.18	1.834	26.0	756.0	1.36	1.40	1.39	1.43
213	S16CS	45.7	16.46	7.486	0.061	42.59	1.425	26.1	733.5	1.33	1.37	1.36	1.39
214	S12CS	26.8	16.46	7.486	0.045	42.95	1.059	26.0	688.1	1.31	1.34	1.37	1.38
215	S10CS	30.6	16.46	7.486	0.037	43.14	0.876	26.1	690.8	1.31	1.34	1.37	1.39
Han and Yan, 2000													
216	SC154-1	59.5	3.84	4.255	0.177	11.95	2.27	163.8	77.0	1.62	1.83	1.74	1.74
217	SC154-2	59.5	3.84	4.255	0.177	11.95	2.27	163.8	65.7	1.39	1.56	1.48	1.48
218	SC154-3	59.5	5.66	4.255	0.177	11.95	2.27	163.8	67.1	1.35	1.52	1.44	1.44
219	SC154-4	59.5	5.66	4.255	0.177	11.95	2.27	163.8	63.0	1.27	1.43	1.35	1.35
220	SC149-1	59.5	5.66	4.255	0.177	11.95	2.27	158.5	71.6	1.35	1.52	1.44	1.44
221	SC149-2	59.5	5.66	4.255	0.177	11.95	2.27	158.5	72.0	1.36	1.53	1.45	1.45
222	SC141-1	59.5	3.84	4.255	0.177	11.95	2.27	150.0	78.8	1.39	1.57	1.52	1.52
223	SC141-2	59.5	3.84	4.255	0.177	11.95	2.27	150.0	83.3	1.47	1.66	1.61	1.61
224	SC130-1	59.5	3.84	4.255	0.177	11.95	2.27	138.3	90.0	1.38	1.54	1.51	1.51
225	SC130-2	59.5	3.84	4.255	0.177	11.95	2.27	138.3	87.8	1.34	1.51	1.48	1.48
226	SC130-3	59.5	5.66	4.255	0.177	11.95	2.27	138.3	99.0	1.44	1.61	1.57	1.57
Kilpatrick and Rangan, 1997													
227	SC-33	59.5	13.92	3.999	0.095	11.40	1.16	85.7	63.5	0.61	0.68	0.60	0.60

Table A-3 - CCFT Column Database

Col. No.	Spec. No.	Fy (ksi)	f'c (ksi)	D (in)	t (in)	Ac (in^2)	As (in^2)	kl (in)	Pexp (k)	Pexp/Ppred by AISC 1999	AISC 2005	Eurocode 4	Plastic
Matsui et al.,1997													
228	C4-0	51.2	4.62	6.51	0.161	30.06	3.21	26.0	351.5	1.48	1.60	1.29	1.45
229	C8-0	51.2	4.62	6.51	0.161	30.06	3.21	52.1	317.7	1.39	1.50	1.35	1.36
230	C12-0	51.2	4.62	6.51	0.161	30.06	3.21	78.1	308.7	1.43	1.55	1.40	1.40
231	C18-0	51.2	4.62	6.51	0.161	30.06	3.21	117.2	258.1	1.36	1.49	1.34	1.34
232	C24-0	51.2	4.62	6.51	0.161	30.06	3.21	156.2	229.3	1.44	1.61	1.52	1.52
233	C30-0	51.2	4.62	6.51	0.161	30.06	3.21	195.3	176.0	1.40	1.58	1.59	1.59
Giakoumelis and Lam, 2003													
234	C3	49.7	4.55	4.51	0.157	13.80	2.14	11.8	213.0	1.58	1.72	1.27	1.54
235	C4	49.7	13.57	4.51	0.157	13.83	2.15	11.8	293.9	1.31	1.38	1.18	1.32
236	C5a	49.7	5.03	4.51	0.150	13.88	2.06	11.8	208.8	1.53	1.66	1.24	1.50
237	C6a	49.7	14.09	4.50	0.155	13.78	2.11	11.8	305.4	1.34	1.42	1.21	1.36
238	C7	52.9	5.03	4.52	0.193	13.44	2.63	11.8	310.1	1.86	2.04	1.49	1.81
239	C8	52.9	15.21	4.53	0.194	13.47	2.64	11.8	401.6	1.52	1.61	1.35	1.52
240	C9	52.9	8.35	4.53	0.198	13.42	2.69	11.8	317.5	1.58	1.71	1.32	1.56
241	C10a	49.7	8.35	4.51	0.148	13.94	2.02	11.8	233.3	1.38	1.48	1.19	1.38
242	C11	49.7	8.35	4.50	0.148	13.88	2.02	11.8	239.8	1.43	1.53	1.23	1.42
243	C12	49.7	4.63	4.50	0.152	13.83	2.07	11.8	224.3	1.68	1.84	1.36	1.65
244	C13	49.7	4.63	4.49	0.152	13.78	2.07	11.8	213.0	1.60	1.75	1.30	1.57
245	C14	49.7	14.34	4.51	0.151	13.90	2.07	11.8	305.4	1.33	1.40	1.21	1.35
246	C15a	49.7	14.34	4.50	0.152	13.85	2.07	11.8	265.6	1.16	1.22	1.05	1.17
Scheneider, 1998													
247	C1	41.3	4.09	5.54	0.118	22.12	2.01	24.0	198.0	1.47	1.58	1.31	1.46
248	C2	45.4	3.45	5.57	0.256	20.07	4.27	24.0	410.1	1.93	2.13	1.61	1.84
249	C3	77.9	4.09	5.51	0.263	19.52	4.34	23.9	610.1	1.80	2.00	1.54	1.70
Han and Yao, 2003													
250	S-1	49.3	2.91	4.72	0.104	16.02	1.51	14.2	143.8	1.49	1.62	1.21	1.45

Table A-3 - CCFT Column Database

Col. No.	Spec. No.	Fy (ksi)	f'c (ksi)	D (in)	t (in)	Ac (in^2)	As (in^2)	kl (in)	Pexp (k)	Pexp/Ppred by AISC 1999	AISC 2005	Eurocode 4	Plastic
251	S-3	49.3	5.22	4.72	0.104	16.02	1.51	14.2	183.4	1.27	1.20	0.95	1.16
252	L-2	49.3	5.22	4.72	0.104	16.02	1.51	55.1	172.8	1.31	1.25	1.19	1.19
Roeder and Cameron, 1999													
253	II -1		6.84	10.28	0.530	66.74	16.23	31.9	109.2	0.11	0.10	0.08	0.10
254	II -2		6.76	10.28	0.530	66.74	16.23	31.9	111.2	0.11	0.10	0.08	0.10
255	II -3		6.76	10.28	0.530	66.74	16.23	31.9	109.7	0.11	0.10	0.08	0.10
256	II -5		6.86	13.72	0.280	136.04	11.82	41.9	70.9	0.06	0.05	0.04	0.05
257	II -6		6.86	13.72	0.280	136.04	11.82	41.9	90.8	0.07	0.07	0.06	0.06
258	II -7		6.37	13.72	0.280	136.04	11.82	69.9	74.6	0.06	0.06	0.05	0.06
259	II -8		6.37	13.72	0.280	136.04	11.82	69.9	79.9	0.07	0.06	0.06	0.06
260	II -9		6.51	23.78	0.220	427.81	16.29	75.9	117.6	0.04	0.04	0.03	0.03
261	II -10		6.84	23.78	0.220	427.81	16.29	75.9	142.9	0.05	0.04	0.04	0.04
O'Shea and Bridge, 2000													
262	S30CL50B	52.7	7.00	6.50	0.111	30.92	2.23	22.1	395.3	1.32	1.24	1.04	1.18
263	S30CL50C	52.7	5.54	6.50	0.111	30.92	2.23	22.5	370.6	1.42	1.34	1.11	1.28
264	S20CL50C	37.2	5.54	7.48	0.076	42.17	1.78	26.0	371.2	1.42	1.30	1.14	1.24
265	S16CL50B	44.4	7.00	7.48	0.060	42.55	1.40	25.9	413.7	1.33	1.21	1.08	1.15
266	S12CL50C	26.9	5.54	7.48	0.044	42.91	1.04	25.9	293.9	1.29	1.17	1.06	1.11
267	S10CL50C	30.6	5.54	7.48	0.034	43.15	0.79	25.9	278.7	1.24	1.12	1.02	1.06
268	S30CL80C	52.7	8.18	6.50	0.111	30.92	2.23	22.9	458.4	1.40	1.30	1.11	1.24
269	S20CL80C	37.2	8.18	7.48	0.076	42.17	1.78	25.8	525.4	1.48	1.35	1.20	1.28
270	S16CL80A	44.4	11.63	7.48	0.060	42.55	1.40	25.9	644.9	1.35	1.23	1.11	1.16
271	S12CL80C	26.9	8.18	7.48	0.044	42.91	1.04	26.0	418.4	1.30	1.18	1.07	1.10
272	S10CL80B	30.6	10.83	7.48	0.034	43.15	0.79	25.9	546.7	1.31	1.19	1.09	1.11
273	S10CL80C	30.6	8.18	7.48	0.034	43.15	0.79	26.2	436.0	1.36	1.23	1.13	1.16
274	S30CL10C	52.7	11.18	6.50	0.111	30.92	2.23	22.5	586.1	1.44	1.33	1.16	1.27
275	S20CL10C	37.2	11.18	7.48	0.076	42.17	1.78	25.8	692.8	1.50	1.37	1.23	1.29
276	S16CL10C	44.4	11.18	7.48	0.060	42.55	1.40	25.9	636.0	1.38	1.26	1.13	1.18

Table A-3 - CCFT Column Database

Col. No.	Spec. No.	Fy (ksi)	f'c (ksi)	D (in)	t (in)	Ac (in^2)	As (in^2)	kl (in)	Pexp (k)	Pexp/Ppred by AISC 1999	AISC 2005	Eurocode 4	Plastic
277	S12CL10C	26.9	11.18	7.48	0.044	42.91	1.04	26.1	591.0	1.37	1.24	1.14	1.16
278	S12CL10A	26.9	15.66	7.48	0.044	42.91	1.04	26.0	723.6	1.23	1.11	1.02	1.03
279	S10CL10C	30.6	11.18	7.48	0.034	43.15	0.79	26.1	573.7	1.34	1.21	1.11	1.13
280	S30CS50B	52.7	7.00	6.50	0.111	30.92	2.23	22.9	373.5	1.25	1.17	0.99	1.12
281	S20CS50A	37.2	5.95	7.48	0.076	42.17	1.78	26.1	377.1	1.36	1.25	1.10	1.19
282	S16CS50B	44.4	7.00	7.48	0.060	42.55	1.40	26.2	380.9	1.22	1.12	0.99	1.06
283	S12CS50A	26.9	5.95	7.48	0.044	42.91	1.04	26.2	309.4	1.28	1.16	1.05	1.09
284	S10CS50A	30.6	5.95	7.48	0.034	43.15	0.79	25.9	303.4	1.26	1.15	1.04	1.08
285	S30CS80A	52.7	11.63	6.50	0.111	30.92	2.23	22.9	515.7	1.24	1.14	0.99	1.08
286	S20CS80B	37.2	10.83	7.48	0.076	42.17	1.78	26.1	582.5	1.30	1.18	1.06	1.11
287	S16CS80A	44.4	11.63	7.48	0.060	42.55	1.40	26.1	584.7	1.23	1.12	1.01	1.05
288	S12CS80A	26.9	11.63	7.48	0.044	42.91	1.04	26.1	515.7	1.16	1.05	0.96	0.98
289	S10CS80B	30.6	10.83	7.48	0.034	43.15	0.79	26.1	550.8	1.32	1.20	1.10	1.12
290	S30CS10A	52.7	15.66	6.50	0.111	30.92	2.23	22.7	600.7	1.15	1.06	0.93	1.00
291	S20CS10A	37.2	15.66	7.48	0.076	42.17	1.78	26.0	755.1	1.22	1.11	1.01	1.04
292	S16CS10A	44.4	15.66	7.48	0.060	42.55	1.40	26.0	732.6	1.18	1.08	0.98	1.01
293	S12CS10A	26.9	15.66	7.48	0.044	42.91	1.04	26.0	687.2	1.17	1.05	0.97	0.98
294	S10CS10A	30.6	15.66	7.48	0.034	43.15	0.79	26.1	689.9	1.17	1.06	0.97	0.99
Kang, Lim, and Moon, 2002													
295	KLM2002	49.7	7.86	4.00	0.142	10.85	1.72	12.0	212.7	1.36	1.29	1.03	1.25
296	KLM2002	40.6	7.86	3.00	0.091	6.26	0.83	9.0	98.6	1.32	1.24	1.01	1.19
297	KLM2002	40.2	7.86	3.51	0.094	8.65	1.01	10.5	139.1	1.42	1.33	1.10	1.28
298	KLM2002	52.9	7.86	4.50	0.090	14.65	1.25	13.5	206.6	1.27	1.19	0.98	1.14
299	KLM2002	49.7	6.77	4.00	0.142	10.85	1.72	12.0	156.7	1.07	1.02	0.80	0.99
300	KLM2002	40.6	6.77	3.00	0.091	6.26	0.83	9.0	67.7	0.98	0.92	0.74	0.89
301	KLM2002	40.2	6.77	3.51	0.094	8.65	1.01	10.5	98.0	1.09	1.02	0.83	0.99
302	KLM2002	52.9	6.77	4.50	0.090	14.65	1.25	13.5	150.5	1.01	0.94	0.77	0.91

Table A-3 - CCFT Column Database

Col.	Spec.	Fy	f'c	D	t	Ac	As	kl	Pexp	Pexp/Ppred by			
No.	No.	(ksi)	(ksi)	(in)	(in)	(in^2)	(in^2)	(in)	(k)	AISC 1999	AISC 2005	Eurocode 4	Plastic
Woo and Kim, 2002													
303	WKC2002	55.8	3.89	12.54	0.272	113.02	10.47	14.1	75.7	0.08	0.08	0.05	0.07
304	WKC2002	55.8	3.89	12.54	0.272	113.02	10.47	26.7	153.1	0.16	0.15	0.11	0.15
305	WKC2002	55.8	3.89	12.54	0.272	113.02	10.47	39.2	272.7	0.29	0.27	0.21	0.27
306	WKC2002	55.8	3.89	12.54	0.272	113.02	10.47	26.7	271.0	0.28	0.27	0.20	0.26
307	WKC2002	55.8	3.89	12.54	0.272	113.02	10.47	26.7	332.8	0.35	0.33	0.25	0.33
308	WKC2002	55.8	3.52	12.54	0.272	113.02	10.47	14.1	62.7	0.07	0.07	0.04	0.06
309	WKC2002	55.8	3.52	12.54	0.272	113.02	10.47	26.7	155.7	0.17	0.16	0.12	0.16
310	WKC2002	55.8	3.52	12.54	0.272	113.02	10.47	39.2	231.8	0.25	0.24	0.19	0.24
311	WKC2002	55.8	3.52	12.54	0.272	113.02	10.47	26.7	302.7	0.33	0.32	0.23	0.31
312	WKC2002	55.8	3.52	12.54	0.272	113.02	10.47	26.7	363.4	0.39	0.38	0.28	0.37

Table A-4 - CCFT Beam-Column Database

Col. No.	Spec. No.	Fy (ksi)	f'c (ksi)	D (in)	ts (in)	Ac (in^2)	As (in^2)	kl (in)	Pexp (k)	e(exp) (in)	Pexp/Ppred by AISC 1999	AISC 2005	Eurocode	Plastic
Furlong, 1967														
1		60.0	4.20	4.50	0.125	14.17	1.73	36.0	100.0	1.00	1.34	1.10	1.09	0.94
2		60.0	4.20	4.50	0.125	14.17	1.73	36.0	90.0	1.18	1.31	1.06	1.07	0.90
3		60.0	4.20	4.50	0.125	14.17	1.73	36.0	75.0	1.75	1.36	1.07	1.13	0.94
4		60.0	4.20	4.50	0.125	14.17	1.73	36.0	50.0	1.76	0.91	0.72	0.76	0.63
5		60.0	4.20	4.50	0.125	14.17	1.73	36.0	25.0	5.76	1.08	0.80	1.08	0.85
6		48.0	3.75	6.00	0.061	27.10	1.18	40.0	127.6	0.69	1.71	1.19	1.14	1.03
7		48.0	3.75	6.00	0.061	27.10	1.18	40.0	94.8	1.66	2.07	1.22	1.36	1.12
8		48.0	3.75	6.00	0.061	27.10	1.18	40.0	64.3	2.37	1.80	1.00	1.20	0.98
9		48.0	3.05	6.00	0.061	27.10	1.18	40.0	30.6	4.69	1.50	0.83	1.26	0.97
10		48.0	3.05	6.00	0.061	27.10	1.18	40.0	30.4	4.38	1.41	0.78	1.16	0.89
11		42.0	5.10	5.00	0.095	17.90	1.40	42.0	127.8	0.61	1.75	1.24	1.21	1.06
12		42.0	5.10	5.00	0.095	17.90	1.40	42.0	120.0	0.93	2.02	1.33	1.34	1.15
13		42.0	5.10	5.00	0.095	17.90	1.40	42.0	90.0	1.57	2.05	1.23	1.43	1.13
14		42.0	5.10	5.00	0.095	17.90	1.40	42.0	79.0	1.77	1.95	1.14	1.37	1.09
15		42.0	5.10	5.00	0.095	17.90	1.40	42.0	78.5	1.61	1.82	1.08	1.27	1.01
16		42.0	5.10	5.00	0.095	17.90	1.40	42.0	77.6	1.81	1.94	1.14	1.37	1.09
17		42.0	5.10	5.00	0.095	17.90	1.40	42.0	68.8	2.19	1.97	1.11	1.40	1.12
18		42.0	5.10	5.00	0.095	17.90	1.40	42.0	60.0	2.60	1.94	1.07	1.44	1.12
19		42.0	5.10	5.00	0.095	17.90	1.40	42.0	58.6	2.65	1.93	1.06	1.44	1.12
20		42.0	5.10	5.00	0.095	17.90	1.40	42.0	39.3	3.70	1.68	0.88	1.39	1.05
21		42.0	5.10	5.00	0.095	17.90	1.40	42.0	20.0	7.04	1.47	0.72	1.40	1.10
22		42.0	5.10	5.00	0.095	17.90	1.40	42.0	9.8	13.27	1.29	0.60	1.32	1.06
Neogi, Sen, and Chapman, 1969														
23	M1	44.0	6.45	6.67	0.201	30.86	4.08	131.0	137.3	1.88	1.29	0.83	1.10	0.63
24	M2	44.0	6.27	6.66	0.207	30.64	4.20	131.0	154.9	1.50	1.29	0.86	1.09	0.65
25	M3	42.0	4.93	6.65	0.223	30.23	4.50	131.0	132.4	1.88	1.25	0.88	1.09	0.68
26	M4	42.5	4.41	6.63	0.258	29.36	5.16	131.0	137.9	1.88	1.18	0.88	1.04	0.67
27	M5	44.4	3.71	6.66	0.283	29.17	5.67	131.0	144.1	1.88	1.13	0.89	1.04	0.67

Table A-4 - CCFT Beam-Column Database

Col.	Spec.	Fy	f'c	D	ts	Ac	As	kl	Pexp	e(exp)	Pexp/Ppred by			
No.	No.	(ksi)	(ksi)	(in)	(in)	(in^2)	(in^2)	(in)	(k)	(in)	AISC 1999	AISC 2005	Eurocode	Plastic
28	M6	44.4	3.85	6.66	0.287	29.09	5.75	131.0	163.0	1.50	1.14	0.90	1.04	0.69
29	M7	46.0	3.83	6.65	0.347	27.86	6.87	130.0	167.2	1.88	1.09	0.88	1.02	0.67
30	M8	38.9	4.82	5.52	0.378	17.83	6.11	131.0	121.0	1.25	1.03	0.82	0.99	0.59
31	M9	38.9	3.14	5.52	0.384	17.74	6.20	131.0	121.0	1.25	1.06	0.88	1.02	0.64
32	M10	41.8	4.94	5.55	0.197	20.88	3.31	131.0	92.0	1.25	1.19	0.84	1.10	0.58
33	C5	27.5	9.68	5.01	0.072	18.60	1.12	55.5	213.0	0.25	2.40	1.36	1.39	1.12
34	C6	38.5	9.68	5.00	0.112	17.92	1.72	55.5	230.8	0.25	1.78	1.28	1.23	1.06
35	C7	27.5	8.71	5.00	0.068	18.58	1.05	67.5	187.9	0.25	2.41	1.39	1.46	1.09
36	C8	38.5	8.71	5.00	0.119	17.81	1.82	67.5	177.5	0.25	1.47	1.08	1.06	0.87
37	C9	27.5	5.82	5.01	0.070	18.63	1.09	80.0	78.1	0.63	1.83	0.94	1.15	0.75
38	C10	38.5	5.82	5.00	0.128	17.68	1.96	80.0	116.2	0.63	1.52	1.07	1.14	0.84
39	C11	27.5	6.18	5.00	0.064	18.64	0.99	80.0	75.9	0.88	2.28	0.98	1.29	0.85
40	C12	38.5	6.18	5.00	0.128	17.68	1.96	80.0	111.1	0.88	1.67	1.09	1.26	0.86
Knowles and Park, 1969														
41	1	58.0	6.00	3.50	0.230	7.25	2.36	32.0	124.6	0.30	1.03	0.95	0.93	0.82
42	2	58.0	6.00	3.50	0.230	7.25	2.36	56.0	105.5	0.30	1.02	0.89	0.92	0.71
43	3	58.0	6.00	3.50	0.230	7.25	2.36	32.0	43.8	1.00	0.56	0.50	0.50	0.42
44	4	58.0	6.00	3.50	0.230	7.25	2.36	44.0	43.0	1.00	0.59	0.51	0.53	0.41
45	5	70.0	6.00	3.25	0.055	7.75	0.55	32.0	67.8	0.30	1.47	1.13	1.10	0.96
46	6	70.0	6.00	3.25	0.055	7.75	0.55	32.0	20.0	1.00	0.76	0.52	0.57	0.43
Rangan and Joyce, 1992														
47		31.6	9.77	4.00	0.063	11.79	0.77	31.8	96.7	0.39	2.00	0.99	1.05	0.87
48		31.6	9.77	4.00	0.063	11.79	0.77	31.8	52.8	1.18	2.31	0.78	1.22	0.84
49		31.6	9.77	4.00	0.063	11.79	0.77	51.7	78.7	0.39	1.77	0.87	1.00	0.70
50		31.6	9.77	4.00	0.063	11.79	0.77	51.7	42.7	1.18	1.96	0.67	1.21	0.68
51		31.6	9.77	4.00	0.063	11.79	0.77	61.6	70.8	0.39	1.68	0.83	0.99	0.63
52		31.6	9.77	4.00	0.063	11.79	0.77	71.6	62.9	0.39	1.59	0.79	0.96	0.56
53		31.6	9.77	4.00	0.063	11.79	0.77	71.6	31.5	1.18	1.55	0.54	1.10	0.50

Table A-4 - CCFT Beam-Column Database

Col. No.	Spec. No.	Fy (ksi)	f'c (ksi)	D (in)	ts (in)	Ac (in^2)	As (in^2)	kl (in)	Pexp (k)	e(exp) (in)	Pexp/Ppred by AISC 1999	AISC 2005	Eurocode	Plastic
54		31.6	9.77	4.00	0.063	11.79	0.77	91.4	49.5	0.39	1.45	0.74	0.98	0.44
55		31.6	9.77	4.00	0.063	11.79	0.77	91.4	28.3	1.18	1.53	0.55	1.22	0.45
Cai, 1991														
56	PB1-1	45.5	5.96	6.54	0.197	29.63	3.92	26.2	330.5	0.79	1.67	1.33	1.26	1.19
57	PB1-2	45.5	5.96	6.54	0.197	29.63	3.92	26.2	341.5	0.79	1.73	1.37	1.30	1.23
58	PB1-3	45.5	5.96	6.54	0.197	29.63	3.92	26.2	278.8	1.57	1.96	1.43	1.41	1.27
59	PB1-4	45.5	5.96	6.54	0.197	29.63	3.92	26.2	279.9	1.57	1.96	1.43	1.42	1.27
60	PB1-5	45.5	5.96	6.54	0.197	29.63	3.92	26.2	158.9	3.94	2.05	1.34	1.64	1.37
61	PB1-6	45.5	5.96	6.54	0.197	29.63	3.92	26.2	143.2	3.94	1.84	1.21	1.48	1.23
62	PB2-1	43.2	5.96	6.54	0.197	29.63	3.92	58.9	329.3	0.79	1.82	1.41	1.39	1.22
63	PB2-2	43.2	5.96	6.54	0.197	29.63	3.92	58.9	321.7	0.79	1.78	1.38	1.36	1.19
64	PB2-3	43.2	5.96	6.54	0.197	29.63	3.92	58.9	245.7	1.57	1.88	1.33	1.42	1.15
65	PB2-4	43.2	5.96	6.54	0.197	29.63	3.92	58.9	257.8	1.57	1.97	1.39	1.49	1.21
66	PB2-5	43.2	5.96	6.54	0.197	29.63	3.92	58.9	130.8	3.94	1.81	1.15	1.60	1.17
67	PB2-6	43.2	5.96	6.54	0.197	29.63	3.92	58.9	127.7	3.94	1.77	1.12	1.56	1.14
68	PB3-1	40.2	5.96	6.54	0.197	29.63	3.92	78.3	275.4	0.79	1.67	1.26	1.31	1.05
69	PB3-2	40.2	5.96	6.54	0.197	29.63	3.92	78.3	259.9	0.79	1.58	1.19	1.23	0.99
70	PB3-3	40.2	5.96	6.54	0.197	29.63	3.92	78.3	205.9	1.57	1.73	1.18	1.39	1.00
71	PB3-4	43.2	5.96	6.54	0.197	29.63	3.92	78.3	201.4	1.57	1.61	1.12	1.31	0.94
72	PB3-5	43.2	5.96	6.54	0.197	29.63	3.92	78.3	107.2	3.94	1.53	0.96	1.49	0.96
73	PB3-6	40.2	5.96	6.54	0.197	29.63	3.92	78.3	115.8	3.94	1.75	1.07	1.70	1.08
74	PC1-1	41.4	4.04	6.54	0.197	29.63	3.92	117.7	229.7	0.79	1.70	1.34	1.56	1.05
75	PC1-2	41.9	4.04	6.54	0.197	29.63	3.92	117.7	245.9	0.79	1.81	1.43	1.66	1.11
76	PC1-3	41.4	4.04	6.54	0.197	29.63	3.92	88.2	193.3	1.57	1.75	1.32	1.50	1.09
77	PC1-4	44.1	4.04	6.54	0.197	29.63	3.92	88.2	203.9	1.57	1.76	1.35	1.53	1.10
78	PC1-5	42.1	5.96	6.54	0.197	29.63	3.92	52.4	328.2	0.79	1.82	1.41	1.37	1.23
79	PC1-6	42.1	5.96	6.54	0.197	29.63	3.92	52.4	352.5	0.79	1.96	1.51	1.47	1.32
80	PC1-7	42.1	5.96	6.54	0.197	29.63	3.92	52.4	147.7	3.94	2.08	1.31	1.77	1.34

Table A-4 - CCFT Beam-Column Database

Col. No.	Spec. No.	Fy (ksi)	f'c (ksi)	D (in)	ts (in)	Ac (in^2)	As (in^2)	kl (in)	Pexp (k)	e(exp) (in)	Pexp/Ppred by AISC 1999	AISC 2005	Eurocode	Plastic
81	PC1-8	42.1	5.96	6.54	0.197	29.63	3.92	78.3	181.9	2.36	1.87	1.23	1.62	1.08
82	PC1-9	42.1	5.96	6.54	0.197	29.63	3.92	78.3	198.3	2.36	2.04	1.34	1.76	1.18
Kloppel and Goder, 1957														
83	7	39.8	2.94	3.74	0.492	5.97	5.02	55.9	212.9	0.05	1.20	1.18	1.15	1.01
84	8	39.5	2.94	3.74	0.502	5.88	5.11	55.9	210.8	0.09	1.22	1.19	1.18	1.02
85	9	39.5	2.94	3.74	0.488	6.00	4.99	55.9	203.9	0.07	1.19	1.16	1.14	1.00
86	10	39.8	2.94	3.74	0.496	5.93	5.05	33.9	228.8	0.08	1.20	1.18	1.16	0.99
87	11	39.5	2.94	3.74	0.500	5.90	5.09	33.9	226.6	0.06	1.16	1.15	1.13	0.96
88	12	39.5	2.94	3.74	0.500	5.90	5.09	33.9	232.4	0.09	1.23	1.21	1.19	1.02
89	13	40.4	2.94	3.74	0.504	5.86	5.12	78.0	199.3	0.14	1.38	1.30	1.33	0.98
90	14	40.0	2.94	3.74	0.492	5.97	5.02	78.0	203.9	0.07	1.35	1.30	1.30	0.98
91	15	40.5	2.94	3.74	0.496	5.93	5.05	78.0	206.1	0.09	1.37	1.31	1.32	0.99
92	41	47.4	3.62	3.74	0.144	9.36	1.63	33.9	147.5	0.07	1.58	1.50	1.45	1.23
93	42	56.0	3.62	3.74	0.145	9.35	1.64	33.9	154.3	0.02	1.40	1.35	1.32	1.09
94	43	48.6	3.62	3.74	0.134	9.47	1.52	33.9	147.5	0.04	1.58	1.51	1.46	1.22
95	44	47.4	3.62	3.74	0.152	9.27	1.71	55.9	127.4	0.02	1.38	1.33	1.28	1.13
96	45	56.0	3.62	3.74	0.154	9.25	1.73	55.9	136.2	0.09	1.42	1.33	1.32	1.10
97	46	48.6	3.62	3.74	0.141	9.39	1.59	55.9	129.4	0.04	1.48	1.41	1.37	1.19
98	47	47.4	3.62	3.74	0.148	9.32	1.67	78.0	120.6	0.11	1.71	1.55	1.59	1.14
99	48	56.0	3.62	3.74	0.149	9.30	1.68	78.0	127.2	0.07	1.56	1.45	1.49	1.04
100	49	48.6	3.62	3.74	0.138	9.42	1.56	78.0	109.6	0.13	1.65	1.47	1.53	1.08
101	63	41.3	3.32	8.50	0.160	52.60	4.19	87.4	230.0	0.13	0.83	0.76	0.74	0.63
102	64	43.4	3.32	8.50	0.162	52.55	4.25	87.4	412.4	0.13	1.43	1.32	1.28	1.09
103	65	41.8	4.32	8.50	0.159	52.63	4.17	87.4	514.6	0.08	1.59	1.46	1.41	1.21
104	66	41.5	4.32	8.50	0.162	52.55	4.25	87.4	503.3	0.17	1.64	1.46	1.41	1.23
105	69	56.5	3.32	8.50	0.238	50.62	6.18	87.4	553.4	0.11	1.28	1.22	1.21	0.99
106	70	57.0	3.32	8.50	0.235	50.69	6.10	87.4	544.3	0.21	1.32	1.24	1.23	1.02
107	71	42.8	4.32	8.50	0.256	50.17	6.63	87.4	630.3	0.06	1.49	1.41	1.37	1.16
108	72	58.7	4.32	8.50	0.248	50.37	6.43	87.4	659.2	0.07	1.32	1.26	1.23	1.01

Table A-4 - CCFT Beam-Column Database

Col.	Spec.	Fy	f'c	D	ts	Ac	As	kl	Pexp	e(exp)	Pexp/Ppred by			
No.	No.	(ksi)	(ksi)	(in)	(in)	(in^2)	(in^2)	(in)	(k)	(in)	AISC 1999	AISC 2005	Eurocode	Plastic
109	73	48.2	3.49	3.74	0.152	9.27	1.71	78.0	112.0	0.06	1.47	1.37	1.39	1.01
110	74	48.9	3.49	3.74	0.134	9.47	1.52	78.0	106.3	0.06	1.51	1.40	1.43	1.03
111	75	51.5	3.49	3.74	0.141	9.39	1.59	78.0	106.3	0.01	1.32	1.28	1.27	0.93
112	76	47.4	3.49	3.74	0.147	9.33	1.66	78.0	92.8	0.12	1.35	1.22	1.26	0.90
113	83	42.8	3.06	4.76	0.144	15.74	2.09	41.3	156.3	0.09	1.36	1.28	1.23	1.05
114	84	47.5	3.06	4.76	0.147	15.69	2.13	41.3	167.8	0.21	1.47	1.36	1.32	1.13
115	85	44.7	3.51	4.76	0.148	15.68	2.15	41.3	188.1	0.16	1.59	1.47	1.42	1.22
116	86	47.4	3.51	4.76	0.157	15.55	2.27	41.3	194.9	0.18	1.55	1.43	1.39	1.19
117	89	49.9	3.06	4.76	0.221	14.67	3.15	41.3	224.4	0.18	1.38	1.31	1.28	1.08
118	90	49.8	3.06	4.76	0.213	14.78	3.05	41.3	228.8	0.06	1.33	1.29	1.24	1.04
119	91	47.9	3.51	4.76	0.215	14.75	3.07	41.3	247.1	0.25	1.65	1.54	1.50	1.29
120	92	46.7	3.51	4.76	0.219	14.70	3.13	41.3	242.5	0.21	1.58	1.49	1.45	1.24
121	95	42.8	3.06	4.76	0.146	15.71	2.12	91.0	144.0	0.02	1.38	1.33	1.29	1.05
122	96	47.5	3.06	4.76	0.148	15.68	2.15	91.0	141.5	0.10	1.38	1.28	1.28	0.99
123	97	44.7	3.51	4.76	0.146	15.71	2.12	91.0	156.3	0.06	1.48	1.39	1.37	1.08
124	98	47.4	3.51	4.76	0.152	15.62	2.20	91.0	169.8	0.07	1.53	1.43	1.42	1.10
125	101	49.9	3.06	4.76	0.224	14.63	3.19	91.0	176.8	0.14	1.30	1.20	1.22	0.93
126	102	49.8	3.06	4.76	0.216	14.74	3.09	91.0	183.4	0.04	1.26	1.22	1.20	0.94
127	103	47.9	3.51	4.76	0.222	14.66	3.17	91.0	196.4	0.04	1.33	1.27	1.25	0.99
128	104	46.7	3.51	4.76	0.214	14.77	3.06	91.0	194.5	0.06	1.40	1.32	1.31	1.03
Kvedaras and Tomaszewicz, 1994														
129	S1	34.8	10.53	9.85	0.079	73.78	2.419	86.7	585.0	0.906	2.75	0.99	1.11	0.89
130	S2	34.8	10.31	9.85	0.079	73.78	2.419	86.7	450.0	1.812	3.55	0.93	1.18	0.94
131	S3	34.8	10.14	9.85	0.079	73.78	2.419	86.7	540.0	0.709	2.20	0.90	0.97	0.78
132	S4	34.8	5.90	9.85	0.079	73.78	2.419	86.7	382.5	1.182	2.50	1.11	1.27	1.01
133	S5	34.8	5.18	9.85	0.079	73.78	2.419	86.7	292.5	1.182	1.99	0.94	1.05	0.84
Johansson, Claeson, Gylltoft, and Akesson, 2000														
134	LFE	62.8	9.35	6.26	0.177	27.43	3.391	106.2	276.8	0.394	1.21	0.94	0.98	0.67

Table A-4 - CCFT Beam-Column Database

Col. No.	Spec. No.	Fy (ksi)	f'c (ksi)	D (in)	ts (in)	Ac (in^2)	As (in^2)	kl (in)	Pexp (k)	e(exp) (in)	Pexp/Ppred by AISC 1999	AISC 2005	Eurocode	Plastic
Kilpatrick and Rangan, 1997														
135	SC-16	59.5	13.92	4.00	0.095	11.40	1.16	85.7	35.3	1.970	1.30	0.65	1.04	0.47
136	SC-33	59.5	13.92	4.00	0.095	11.40	1.16	85.7	63.5	0.788	1.40	0.88	1.00	0.44
Matsui et al.,1997														
137	C4-1	51.2	4.62	6.51	0.161	30.06	3.21	26.0	272.9	0.816	2.21	1.35	1.40	1.12
138	C4-3	51.2	4.62	6.51	0.161	30.06	3.21	26.0	169.7	2.443	2.19	1.29	1.44	1.12
139	C4-5	51.2	4.62	6.51	0.161	30.06	3.21	26.0	124.7	4.066	2.17	1.27	1.44	1.12
140	C8-1	51.2	4.62	6.51	0.161	30.06	3.21	52.1	234.0	0.816	1.88	1.04	1.39	1.05
141	C8-3	51.2	4.62	6.51	0.161	30.06	3.21	52.1	148.1	2.443	1.64	0.83	1.40	1.10
142	C8-5	51.2	4.62	6.51	0.161	30.06	3.21	52.1	97.7	4.066	1.77	1.53	1.13	1.08
143	C12-1	51.2	4.62	6.51	0.161	30.06	3.21	78.1	213.1	0.816	1.92	1.63	1.28	1.06
144	C12-3	51.2	4.62	6.51	0.161	30.06	3.21	78.1	128.5	2.443	1.93	1.43	1.29	0.98
145	C12-5	51.2	4.62	6.51	0.161	30.06	3.21	78.1	86.9	4.066	2.12	1.57	1.58	1.28
146	C18-1	51.2	4.62	6.51	0.161	30.06	3.21	117.2	166.7	0.816	2.41	1.53	1.72	1.33
147	C18-3	51.2	4.62	6.51	0.161	30.06	3.21	117.2	103.5	2.443	2.06	1.20	1.15	0.75
148	C18-5	51.2	4.62	6.51	0.161	30.06	3.21	117.2	74.3	4.066	1.73	1.37	1.14	0.84
149	C24-1	51.2	4.62	6.51	0.161	30.06	3.21	156.2	137.0	0.816	1.19	1.22	0.93	0.84
150	C24-3	51.2	4.62	6.51	0.161	30.06	3.21	156.2	78.8	2.443	1.16	1.16	0.92	0.71
151	C24-5	51.2	4.62	6.51	0.161	30.06	3.21	156.2	62.3	4.066	1.68	1.46	1.10	0.96
152	C30-1	51.2	4.62	6.51	0.161	30.06	3.21	195.6	107.8	0.816	2.55	1.24	1.29	0.85
153	C30-3	51.2	4.62	6.51	0.161	30.06	3.21	195.6	69.5	2.443	1.88	1.37	1.26	0.86
154	C30-5	51.2	4.62	6.51	0.161	30.06	3.21	195.6	53.6	4.066	1.44	1.08	1.09	0.65
Elremaily and Azizinamini, 2002														
155	CFT1	54.2	14.50	12.76	0.252	117.90	9.90	86.0	137.5	6.47	1.45	1.03	1.10	0.63
156	CFT2	53.8	15.08	12.76	0.374	113.25	14.55	86.0	222.9	10.96	1.75	1.61	1.11	1.08
157	CFT3	53.8	15.08	12.76	0.374	113.25	14.55	86.0	108.2	5.89	1.71	1.33	0.99	0.93
158	CFT4	53.8	5.80	12.76	0.374	113.25	14.55	86.0	360.9	7.97	2.74	1.81	1.46	1.09

Table A-4 - CCFT Beam-Column Database

Col. No.	Spec. No.	Fy (ksi)	f'c (ksi)	D (in)	ts (in)	Ac (in^2)	As (in^2)	kl (in)	Pexp (k)	e(exp) (in)	Pexp/Ppred by AISC 1999	AISC 2005	Eurocode	Plastic
159	CFT5	54.2	5.80	12.76	0.252	117.90	9.90	86.0	157.6	8.17	2.14	1.29	1.72	1.33
160	CFT6	54.2	10.15	12.76	0.252	117.90	9.90	86.0	167.1	8.16	2.29	1.08	1.79	1.34
Han and Yao, 2003														
161	S-2	49.3	2.91	4.72	0.104	16.02	1.51	14.2	119.8	0.55	1.59	1.38	1.32	1.26
162	S-4	49.3	5.22	4.72	0.104	16.02	1.51	14.2	134.8	0.55	1.54	1.20	1.13	1.08
163	L-1	49.3	5.22	4.72	0.104	16.02	1.51	55.1	132.6	0.55	1.68	1.27	1.28	1.06
164	L-5	49.3	5.22	4.72	0.104	16.02	1.51	55.1	92.6	1.22	1.71	1.16	1.29	0.98
O'Shea and Bridge, 2000														
165	S30E250B	52.7	7.00	6.50	0.111	30.92	2.23	22.9	342.7	0.28	1.51	1.23	1.11	1.08
166	S20E250A	37.2	5.95	7.48	0.076	42.17	1.78	26.0	344.5	0.34	1.92	1.33	1.21	1.18
167	S12E250A	26.9	5.95	7.48	0.044	42.91	1.04	26.1	276.2	0.33	2.39	1.21	1.10	1.06
168	S10E250A	30.6	5.95	7.48	0.034	43.15	0.79	26.1	273.9	0.29	2.39	1.20	1.08	1.05
169	S30E150B	52.7	7.00	6.50	0.111	30.92	2.23	22.8	252.4	0.68	1.49	1.04	0.99	0.93
170	S20E150A	37.2	5.95	7.48	0.076	42.17	1.78	26.1	288.5	0.64	2.11	1.22	1.14	1.09
171	S16E150B	44.4	7.00	7.48	0.060	42.55	1.40	26.1	283.1	0.61	1.97	1.06	1.00	0.93
172	S12E150A	26.9	5.95	7.48	0.044	42.91	1.04	26.1	229.9	0.74	3.26	1.15	1.19	1.09
173	S10E150A	30.6	5.95	7.48	0.034	43.15	0.79	26.1	228.5	0.55	2.91	1.10	1.07	1.00
174	S30E280A	52.7	11.63	6.50	0.111	30.92	2.23	22.8	436.0	0.37	1.66	1.15	1.06	1.02
175	S20E280B	37.2	10.83	7.48	0.076	42.17	1.78	26.1	495.1	0.39	2.23	1.19	1.09	1.05
176	S10E280B	30.6	10.83	7.48	0.034	43.15	0.79	26.2	429.2	0.34	3.31	1.13	1.05	0.99
177	S30E180A	52.7	11.63	6.50	0.111	30.92	2.23	22.8	371.5	0.70	1.88	1.09	1.04	0.98
178	S20E180B	37.2	10.83	7.48	0.076	42.17	1.78	26.1	388.8	0.82	2.70	1.05	1.11	1.01
179	S16E180A	44.4	11.63	7.48	0.060	42.55	1.40	26.1	432.6	0.56	2.41	1.05	1.01	0.94
180	S10E180B	30.6	10.83	7.48	0.034	43.15	0.79	26.2	344.3	0.70	4.61	1.04	1.07	0.96
181	S30E210B	52.7	16.34	6.50	0.111	30.92	2.23	22.8	504.7	0.27	1.45	0.98	0.90	0.87
182	S20E210B	37.2	16.34	7.48	0.076	42.17	1.78	26.0	602.9	0.26	1.84	0.96	0.87	0.85
183	S10E210B	30.6	16.34	7.48	0.034	43.15	0.79	26.0	474.6	0.16	1.95	0.78	0.70	0.67

Table A-4 - CCFT Beam-Column Database

Col. No.	Spec. No.	Fy (ksi)	f'c (ksi)	D (in)	ts (in)	Ac (in^2)	As (in^2)	kl (in)	Pexp (k)	e(exp) (in)	Pexp/Ppred by AISC 1999	AISC 2005	Eurocode	Plastic
184	S30E110B	52.7	16.34	6.50	0.111	30.92	2.23	22.8	422.5	0.61	2.01	1.19	0.90	0.83
185	S20E110B	37.2	16.34	7.48	0.076	42.17	1.78	26.2	536.2	0.67	3.28	1.24	1.03	0.93
186	S16E110B	44.4	16.34	7.48	0.060	42.55	1.40	26.0	543.8	0.51	2.88	1.25	0.94	0.87
187	S12E110B	26.9	16.34	7.48	0.044	42.91	1.04	26.1	432.6	0.67	5.19	1.15	0.90	0.81
Jung et al.,1994														
188	5	35.4	4.79	10.53	0.207	80.34	6.70	51.2	224.7	2.47	1.19	0.81	0.68	0.58
189	6	35.4	4.79	10.53	0.207	80.34	6.70	51.2	224.7	2.41	1.17	0.80	0.67	0.58
190	7	35.4	4.79	10.53	0.207	80.34	6.70	51.2	337.1	1.76	1.48	1.08	0.83	0.77
191	8	35.4	4.79	10.53	0.207	80.34	6.70	51.2	224.7	2.65	1.24	0.83	0.71	0.61
192	9	35.4	4.79	10.53	0.207	80.34	6.70	51.2	224.7	2.72	1.26	0.84	0.73	0.62
193	11	35.4	6.38	10.53	0.207	80.34	6.70	51.2	337.1	1.84	1.40	0.91	0.74	0.65
194	12	35.4	6.38	10.53	0.207	80.34	6.70	51.2	224.7	2.92	1.24	0.72	0.67	0.57
195	17	35.4	4.79	10.53	0.157	81.91	5.13	51.2	224.7	1.94	1.25	0.81	0.66	0.58
196	18	35.4	4.79	10.53	0.157	81.91	5.13	51.2	337.1	1.31	1.52	1.09	0.82	0.76
197	20	35.4	6.38	10.53	0.157	81.91	5.13	51.2	224.7	1.95	1.16	0.66	0.57	0.50
198	21	35.4	6.38	10.53	0.157	81.91	5.13	51.2	337.1	1.30	1.38	0.89	0.68	0.62

Table A-5 - RCFT Column Database

Col. No.	Spec. No.	Fy (ksi)	f'c (ksi)	h1 (in)	h2 (in)	t (in)	Ac (in^2)	As (in^2)	kl (in)	Pexp (k)	Pexp/Ppred by AISC 1999	AISC 2005	Eurocode	Plastic
Furlong, 1967														
1		70.3	6.50	5.00	5.00	0.189	21.36	3.64	36.0	360.0	1.00	1.00	0.93	0.91
2		48.0	3.40	4.00	4.00	0.084	14.68	1.32	36.0	117.8	1.16	1.16	1.07	1.04
3		48.0	3.40	4.00	4.00	0.084	14.68	1.32	36.0	109.8	1.08	1.08	1.00	0.97
4		48.0	4.18	4.00	4.00	0.125	14.06	1.94	36.0	150.0	1.09	1.09	1.02	0.99
5		48.0	4.18	4.00	4.00	0.125	14.06	1.94	36.0	152.0	1.11	1.11	1.03	1.00
Chapman and Neogi, 1966														
6	DF3	36.9	4.66	4.50	4.50	0.379	14.00	6.25	16.0	549.0	1.93	1.93	1.86	1.86
7	DF4	36.9	4.66	4.52	4.52	0.173	17.42	3.01	16.0	201.6	1.13	1.13	1.05	1.05
Knowles and Park, 1969														
8		47.0	5.20	3.00	3.00	0.131	7.50	1.50	68.0	80.0	0.99	0.99	0.92	0.73
9		47.0	6.78	3.00	3.00	0.131	7.50	1.50	44.0	95.0	0.93	0.93	0.86	0.78
10		47.0	4.94	3.00	3.00	0.133	7.47	1.53	56.0	86.6	0.99	0.99	0.92	0.80
11		47.0	6.54	3.00	3.00	0.133	7.47	1.53	32.0	104.0	0.97	0.97	0.90	0.86
12		47.0	5.93	3.00	3.00	0.133	7.47	1.53	20.0	113.7	1.06	1.06	0.99	0.98
13		47.0	5.93	3.00	3.00	0.133	7.47	1.53	10.0	115.0	1.06	1.06	0.99	0.99
Bridge, 1976														
14	SCH-2	42.1	4.44	7.87	7.87	0.394	50.22	11.78	120.1	645.0	1.05	1.05	0.98	0.90
Shakir-Khalil and Zeghiche, 1989														
15	1	56.0	5.10	4.72	3.15	0.197	11.93	2.92	115.7	134.9	1.30	1.31	1.40	0.60
Shakir-Khalil and Mouli, 1990														
16	6	50.3	5.58	5.91	3.94	0.197	18.82	3.47	115.8	225.5	1.32	1.30	1.27	0.81
Janss, 1974														
17	21	53.7	4.58	12.99	12.99	0.176	159.77	9.02	51.9	980.9	0.89	0.90	0.81	0.81

Table A-5 - RCFT Column Database

Col. No.	Spec. No.	Fy (ksi)	f'c (ksi)	h1 (in)	h2 (in)	t (in)	Ac (in^2)	As (in^2)	kl (in)	Pexp (k)	Pexp/Ppred by AISC 1999	AISC 2005	Eurocode	Plastic
18	22	53.7	3.97	13.03	13.03	0.176	160.76	9.05	52.3	991.8	0.97	0.97	0.88	0.88
19	23	53.7	3.97	13.03	13.03	0.177	160.71	9.10	52.0	1046.9	1.02	1.02	0.93	0.93
20	24	53.7	4.58	13.03	13.03	0.177	160.71	9.10	51.9	991.8	0.90	0.90	0.81	0.81
21	25	64.5	4.58	13.11	13.11	0.251	158.96	12.91	51.9	1318.0	0.92	0.92	0.84	0.84
22	26	64.5	4.58	13.03	13.03	0.248	157.13	12.68	51.9	1313.6	0.93	0.93	0.85	0.85
23	27	64.5	3.97	13.03	13.03	0.249	157.08	12.73	51.9	1311.4	0.98	0.98	0.91	0.91
24	28	64.5	3.97	13.03	13.03	0.249	157.08	12.73	52.0	1267.3	0.95	0.95	0.88	0.88
25	29	56.5	4.14	13.03	13.03	0.398	149.70	20.11	55.0	1818.3	1.10	1.10	1.04	1.04
26	30	56.5	4.14	12.95	12.95	0.402	147.60	20.18	55.0	1829.3	1.11	1.11	1.04	1.04
27	31	56.5	4.14	12.99	12.99	0.398	148.74	20.05	55.0	1796.3	1.10	1.10	1.03	1.03
28	32	56.5	4.14	13.11	13.11	0.398	151.63	20.24	55.0	1829.3	1.10	1.10	1.03	1.03
Baba, Fujimoto, Mukai, and Nishiyama, 1995														
29	CR4-A-2	37.9	3.68	5.84	5.84	0.172	30.22	3.91	8.8	259.2	1.07	1.07	1.00	1.00
30	CR4-A-4-1	37.9	5.86	5.83	5.83	0.172	30.14	3.91	8.8	317.8	1.07	1.07	0.98	0.98
31	CR4-A-4-2	37.9	5.86	5.83	5.83	0.172	30.14	3.91	8.8	315.2	1.06	1.06	0.97	0.97
32	CR4-A-8	37.9	11.14	5.83	5.83	0.172	30.14	3.91	8.8	473.9	1.09	1.09	0.98	0.98
33	CR4-C-2	37.9	3.68	8.47	8.47	0.172	66.06	5.73	12.7	399.4	0.94	0.94	0.87	0.87
34	CR4-C-4-1	37.9	5.95	8.46	8.46	0.172	65.87	5.72	12.7	544.8	0.99	0.99	0.90	0.90
35	CR4-C-4-2	37.9	5.95	8.46	8.46	0.172	65.80	5.71	12.7	537.8	0.98	0.98	0.88	0.88
36	CR4-C-8	37.9	11.62	8.46	8.46	0.172	65.80	5.71	12.7	862.4	1.00	1.00	0.88	0.88
37	CR4-D-2	37.9	3.68	12.72	12.72	0.172	153.06	8.65	19.1	756.6	0.94	0.94	0.85	0.85
38	CR4-D-4-1	37.9	5.95	12.72	12.72	0.172	153.06	8.65	19.1	1112.6	1.01	1.01	0.90	0.90
39	CR4-D-4-2	37.9	5.95	12.72	12.72	0.172	153.06	8.65	19.1	1085.5	0.99	0.99	0.88	0.88
40	CR4-D-8	37.9	11.62	12.74	12.74	0.172	153.64	8.67	19.1	1681.2	0.91	0.91	0.80	0.80
41	CR6-A-2	89.4	3.68	5.68	5.68	0.250	26.79	5.44	8.5	578.1	1.02	1.02	0.99	0.99
42	CR6-A-4-1	89.4	5.86	5.67	5.67	0.250	26.75	5.43	8.5	631.0	1.02	1.02	0.98	0.98
43	CR6-A-4-2	89.4	5.86	5.67	5.67	0.250	26.75	5.43	8.5	621.5	1.01	1.01	0.97	0.97
44	CR6-A-8	89.4	11.14	5.67	5.67	0.250	26.67	5.42	8.5	763.9	1.04	1.04	0.98	0.98
45	CR6-C-2	89.4	3.68	8.30	8.30	0.250	60.82	8.06	12.4	880.9	0.97	0.97	0.93	0.93

Table A-5 - RCFT Column Database

Col. No.	Spec. No.	Fy (ksi)	f'c (ksi)	h1 (in)	h2 (in)	t (in)	Ac (in^2)	As (in^2)	kl (in)	Pexp (k)	Pexp/Ppred by AISC 1999	AISC 2005	Eurocode	Plastic
46	CR6-C-4-1	89.4	5.86	8.31	8.31	0.250	60.94	8.07	12.5	995.1	0.97	0.97	0.92	0.92
47	CR6-C-4-2	89.4	5.86	8.30	8.30	0.250	60.82	8.06	12.4	1007.7	0.99	0.99	0.94	0.94
48	CR6-C-8	89.4	11.14	8.29	8.29	0.250	60.69	8.05	12.4	1294.2	1.00	1.00	0.93	0.93
49	CR6-D-2	89.4	3.68	12.56	12.56	0.250	145.31	12.32	18.8	1420.5	0.91	0.91	0.87	0.87
50	CR6-D-4-1	89.4	5.95	12.54	12.54	0.250	145.02	12.31	18.8	1748.4	0.95	0.96	0.89	0.89
51	CR6-D-4-2	89.4	5.95	12.53	12.53	0.250	144.64	12.30	18.8	1679.4	0.92	0.92	0.86	0.86
52	CR6-D-8	89.4	12.32	12.54	12.54	0.250	145.02	12.31	18.8	2327.6	0.89	0.89	0.81	0.81
53	CR8-A-2	120.8	3.68	4.72	4.72	0.255	17.77	4.55	7.1	633.7	1.05	1.05	1.03	1.03
54	CR8-A-4-1	120.8	5.86	4.74	4.74	0.255	17.90	4.57	7.1	664.5	1.04	1.04	1.01	1.01
55	CR8-A-4-2	120.8	5.86	4.74	4.74	0.255	17.90	4.57	7.1	665.4	1.04	1.04	1.01	1.01
56	CR8-A-8	120.8	11.14	4.70	4.70	0.255	17.53	4.53	7.0	745.6	1.05	1.05	1.00	1.00
57	CR8-C-2	120.8	3.68	6.89	6.89	0.255	40.71	6.76	10.3	946.2	1.00	1.00	0.98	0.98
58	CR8-C-4-1	120.8	5.86	6.88	6.88	0.255	40.61	6.75	10.3	1009.9	0.99	0.99	0.96	0.96
59	CR8-C-4-2	120.8	5.86	6.89	6.89	0.255	40.66	6.76	10.3	1020.9	1.00	1.00	0.97	0.97
60	CR8-C-8	120.8	11.14	6.89	6.89	0.255	40.66	6.76	10.3	1206.0	1.01	1.01	0.95	0.95
61	CR8-D-2	120.8	3.68	10.43	10.43	0.255	98.32	10.36	15.6	1471.2	0.95	0.95	0.91	0.91
62	CR8-D-4-1	120.8	5.95	10.39	10.39	0.255	97.54	10.32	15.6	1599.4	0.92	0.92	0.88	0.88
63	CR8-D-4-2	120.8	5.95	10.41	10.41	0.255	98.09	10.35	15.6	1611.8	0.93	0.93	0.88	0.88
64	CR8-D-8	120.8	11.62	10.43	10.43	0.255	98.40	10.37	15.6	2020.4	0.91	0.91	0.84	0.84
Grauers, 1993														
65	1	44.1	6.82	4.72	4.72	0.197	18.76	3.57	9.8	323.7	1.22	1.22	1.14	1.14
66	2	63.5	6.67	4.72	4.72	0.197	18.76	3.57	9.8	379.9	1.14	1.14	1.08	1.08
67	3	47.4	13.92	4.72	4.72	0.197	18.76	3.57	9.8	458.6	1.18	1.18	1.07	1.07
68	4	63.7	13.92	4.72	4.72	0.197	18.76	3.57	9.8	503.6	1.13	1.13	1.03	1.03
69	5	46.8	5.66	4.72	4.72	0.315	16.76	5.56	9.8	348.5	1.02	1.02	0.98	0.98
70	6	43.5	6.67	4.72	4.72	0.315	16.76	5.56	9.8	375.4	1.12	1.12	1.06	1.06
71	7	54.5	6.82	4.72	4.72	0.315	16.76	5.56	9.8	447.4	1.12	1.12	1.07	1.07
72	8	46.8	14.94	4.72	4.72	0.315	16.76	5.56	9.8	510.3	1.08	1.08	1.00	1.00
73	9	55.0	14.94	4.72	4.72	0.315	16.76	5.56	9.8	602.5	1.17	1.17	1.08	1.08

Table A-5 - RCFT Column Database

Col. No.	Spec. No.	Fy (ksi)	f'c (ksi)	h1 (in)	h2 (in)	t (in)	Ac (in^2)	As (in^2)	kl (in)	Pexp (k)	Pexp/Ppred by AISC 1999	AISC 2005	Eurocode	Plastic
74	10	55.0	5.66	4.72	4.72	0.315	16.76	5.56	9.8	404.7	1.05	1.05	1.01	1.01
75	11	54.5	13.49	4.72	4.72	0.315	16.76	5.56	9.8	634.0	1.28	1.28	1.20	1.20
76	12	52.8	13.49	4.72	4.72	0.315	16.76	5.56	9.8	609.3	1.26	1.26	1.17	1.17
77	13	52.8	11.60	4.72	4.72	0.315	16.76	5.56	9.8	517.1	1.13	1.13	1.06	1.06
78	14	55.0	11.60	4.72	4.72	0.315	16.76	5.56	9.8	514.8	1.10	1.10	1.03	1.03
79	15s	57.4	13.92	4.72	4.72	0.315	16.76	5.56	9.8	526.1	1.02	1.02	0.95	0.95
80	16	57.4	13.92	4.72	4.72	0.315	16.76	5.56	9.8	260.8	0.51	0.51	0.47	0.47
81	17	58.6	13.34	4.72	4.72	0.315	16.76	5.56	9.8	310.3	0.60	0.60	0.56	0.56
82	18	58.6	13.34	4.72	4.72	0.315	16.76	5.56	9.8	328.2	0.64	0.64	0.60	0.60
83	25	57.4	13.34	4.72	4.72	0.315	16.76	5.56	9.8	517.1	0.69	0.69	0.66	0.37
84	23	55.0	4.50	4.72	4.72	0.315	16.76	5.56	9.8	377.7	0.00	0.00	0.00	0.00
85	24	55.0	13.34	4.72	4.72	0.315	16.76	5.56	9.8	546.3	2.08	2.07	2.26	0.99
86	27	55.0	4.79	9.84	9.84	0.315	84.87	12.00	9.8	1094.9	2.07	2.07	2.26	0.99
87	28	55.0	13.20	9.84	9.84	0.315	84.87	12.00	9.8	1866.0	1.78	1.78	1.93	0.85
88	15c	57.4	13.92	4.72	4.72	0.315	16.76	5.56	129.0	206.8	0.69	0.69	0.66	0.37
Lin, 1988														
89	D7	35.7	3.26	5.91	5.91	0.028	34.23	0.65	18.9	125.4	1.07	1.07	0.93	0.93
90	D8	35.7	3.26	5.91	5.91	0.028	34.23	0.65	31.5	137.5	1.18	1.18	1.02	1.02
91	D10	35.9	3.26	5.91	5.91	0.055	33.59	1.29	31.5	160.0	1.16	1.16	1.03	1.03
92	D12	36.1	3.26	5.91	5.91	0.083	32.95	1.93	31.5	178.3	1.12	1.12	1.01	1.01
93	D13	35.8	3.26	5.91	7.87	0.028	45.74	0.76	18.9	178.7	1.16	1.16	1.01	1.01
94	D14	35.8	3.26	5.91	7.87	0.028	45.74	0.76	31.5	158.5	1.04	1.04	0.90	0.90
95	D16	35.8	3.26	5.91	7.87	0.055	44.99	1.51	31.5	198.1	1.12	1.12	0.99	0.99
96	D18	36.0	3.26	5.91	7.87	0.083	44.25	2.25	31.5	189.8	0.94	0.94	0.84	0.84
97	E7	35.7	4.88	5.91	5.91	0.028	34.23	0.65	18.9	167.9	1.02	1.02	0.88	0.88
98	E10	35.9	5.11	5.91	5.91	0.055	33.59	1.29	31.5	218.9	1.16	1.16	1.01	1.00
99	E15	35.8	4.88	5.91	7.87	0.055	44.99	1.51	18.9	263.6	1.10	1.10	0.96	0.96
100	E18	36.0	5.11	5.91	7.87	0.083	44.25	2.25	31.5	285.0	1.05	1.05	0.93	0.93

Table A-5 - RCFT Column Database

Col. No.	Spec. No.	Fy (ksi)	f'c (ksi)	h1 (in)	h2 (in)	t (in)	Ac (in^2)	As (in^2)	kl (in)	Pexp (k)	Pexp/Ppred by AISC 1999	AISC 2005	Eurocode	Plastic
Song and Kwon, 1997														
101	US 9	45.5	4.37	5.25	5.25	0.126	24.958	2.583	15.4	261.3	1.25	1.25	1.15	1.15
102	US 12	45.5	4.37	7.02	7.02	0.126	45.817	3.477	20.7	366.0	1.12	1.12	1.02	1.02
103	US 15	45.5	4.37	8.77	8.77	0.118	72.788	4.090	26.0	542.4	1.19	1.19	1.08	1.08
Matsui et al.,1997														
104	C4-0	59.7	5.93	5.90	5.90	0.168	30.976	3.858	23.6	359.6	0.94	0.94	0.87	0.87
105	C8-0	59.7	5.93	5.90	5.90	0.168	30.976	3.858	47.3	356.9	0.96	0.96	0.89	0.86
106	C12-0	59.7	5.93	5.90	5.90	0.168	30.976	3.858	70.9	353.9	1.00	1.00	0.92	0.86
107	C18-0	59.7	5.93	5.90	5.90	0.168	30.976	3.858	106.4	305.1	0.97	0.97	0.88	0.74
108	C24-0	59.7	5.93	5.90	5.90	0.168	30.976	3.858	141.8	257.2	0.95	0.96	0.90	0.62
109	C30-0	59.7	5.93	5.90	5.90	0.168	30.976	3.858	177.3	204.5	0.91	0.91	0.92	0.49
Scheneider, 1998														
110	S1	51.6	4.42	5.01	5.01	0.124	22.70	2.38	24.0	206.1	1.00	1.00	0.92	0.92
111	S2	51.8	3.78	5.00	5.00	0.171	21.60	3.22	24.0	246.1	1.05	1.06	0.99	0.99
112	S3	46.7	3.45	5.00	5.00	0.179	21.50	3.37	24.0	250.1	1.15	1.15	1.08	1.08
113	S4	45.2	3.45	4.93	4.98	0.223	20.30	4.16	24.1	270.1	1.10	1.10	1.05	1.05
114	S5	50.3	3.45	4.99	5.01	0.294	19.40	5.31	24.0	464.9	1.45	1.45	1.39	1.39
115	R1	62.4	4.42	3.02	6.00	0.118	16.00	2.04	24.1	184.0	1.04	0.99	0.93	0.93
116	R2	55.5	3.78	3.01	6.02	0.176	15.00	2.98	23.9	226.1	1.12	1.07	1.02	1.02
117	R3	59.9	3.78	4.01	6.00	0.170	20.70	3.22	24.0	257.1	1.02	1.00	0.95	0.95
118	R4	52.9	3.45	4.05	6.01	0.180	20.80	3.41	24.0	275.1	1.17	1.15	1.09	1.09
119	R5	47.0	3.45	3.99	5.96	0.225	19.50	4.14	24.1	300.0	1.22	1.20	1.15	1.15
120	R6	51.9	3.45	4.02	6.00	0.289	18.60	5.24	24.0	380.0	1.19	1.17	1.13	1.13
Han and Yao, 2002														
121	M-1-1	49.3	3.35	5.12	5.12	0.104	24.10	2.09	30.7	170.8	1.01	1.01	0.93	0.93
122	M-1-2	49.3	3.35	5.12	5.12	0.104	24.10	2.09	30.7	173.0	1.02	1.03	0.95	0.94
123	H-1-1	49.3	3.35	5.12	5.12	0.104	24.10	2.09	30.7	155.1	0.92	0.92	0.85	0.84

Table A-5 - RCFT Column Database

Col. No.	Spec. No.	Fy (ksi)	f'c (ksi)	h1 (in)	h2 (in)	t (in)	Ac (in^2)	As (in^2)	kl (in)	Pexp (k)	Pexp/Ppred by AISC 1999	AISC 2005	Eurocode	Plastic
124	H-1-2	49.3	3.35	5.12	5.12	0.104	24.10	2.09	30.7	166.1	0.98	0.98	0.91	0.90
125	M-2-1	49.3	3.35	14.17	9.45	0.104	129.03	4.89	56.7	516.9	0.86	0.87	0.77	0.77
126	M-2-2	49.3	3.35	14.17	9.45	0.104	129.03	4.89	56.7	505.6	0.84	0.85	0.76	0.75
127	H-2-1	49.3	3.35	14.17	9.45	0.104	129.03	4.89	56.7	361.8	0.60	0.61	0.54	0.54
128	H-2-2	49.3	3.35	14.17	9.45	0.104	129.03	4.89	56.7	359.6	0.59	0.60	0.54	0.53
129	M-3-1	49.3	3.35	7.68	5.12	0.104	36.67	2.63	30.7	220.2	0.95	0.96	0.88	0.87
130	M-2-2	49.3	3.35	7.68	5.12	0.104	36.67	2.63	30.7	215.7	0.93	0.94	0.86	0.85
131	H-3-1	49.3	3.35	7.68	5.12	0.104	36.67	2.63	30.7	197.8	0.85	0.86	0.79	0.78
132	H-3-2	49.3	3.35	7.68	5.12	0.104	36.67	2.63	30.7	202.2	0.87	0.88	0.80	0.80
133	M-6-1	49.3	3.35	7.68	5.12	0.104	36.67	2.63	92.1	200.0	0.91	1.00	0.91	0.79
134	M-6-2	49.3	3.35	7.68	5.12	0.104	36.67	2.63	92.1	183.1	0.83	0.91	0.83	0.73
135	H-6-1	49.3	3.35	7.68	5.12	0.104	36.67	2.63	92.1	144.9	0.66	0.72	0.66	0.57
136	H-6-2	49.3	3.35	7.68	5.12	0.104	36.67	2.63	92.1	140.5	0.64	0.70	0.64	0.56
137	M-8-1	49.3	3.35	5.31	3.54	0.104	17.03	1.80	21.3	130.3	0.95	0.96	0.89	0.89
138	M-8-2	49.3	3.35	5.31	3.54	0.104	17.03	1.80	21.3	133.0	0.97	0.98	0.91	0.91
139	H-8-1	49.3	3.35	5.31	3.54	0.104	17.03	1.80	21.3	128.1	0.94	0.95	0.88	0.88
140	H-8-2	49.3	3.35	5.31	3.54	0.104	17.03	1.80	21.3	124.0	0.91	0.92	0.85	0.85
141	M-9-1	49.3	3.35	9.45	4.72	0.104	41.73	2.91	28.3	256.2	0.98	0.99	0.91	0.90
142	M-9-2	49.3	3.35	9.45	4.72	0.104	41.73	2.91	28.3	231.9	0.89	0.90	0.82	0.82
143	H-9-1	49.3	3.35	9.45	4.72	0.104	41.73	2.91	28.3	217.5	0.83	0.84	0.77	0.77
Uy, 2000														
144	HSS1	108.8	4.06	4.33	4.33	0.197	15.50	3.26	118.1	412.6	2.08	2.07	2.26	0.99
145	HSS2	108.8	4.06	4.33	4.33	0.197	15.50	3.26	118.1	411.7	2.07	2.07	2.26	0.99
146	HSS5	108.8	4.35	4.33	4.33	0.197	15.50	3.26	118.1	356.2	1.78	1.78	1.93	0.85
147	HSS8	108.8	4.35	6.30	6.30	0.197	34.88	4.81	118.1	644.5	1.36	1.37	1.32	0.96
148	HSS9	108.8	4.35	6.30	6.30	0.197	34.88	4.81	118.1	656.6	1.39	1.39	1.35	0.97
149	HSS12	108.8	4.35	6.30	6.30	0.197	34.88	4.81	118.1	503.8	1.07	1.07	1.04	0.75
150	HSS14	108.8	5.80	8.27	8.27	0.197	62.00	6.36	118.1	833.7	1.01	1.01	0.93	0.79
151	HSS15	108.8	5.80	8.27	8.27	0.197	62.00	6.36	118.1	782.7	0.94	0.95	0.87	0.74

Table A-5 - RCFT Column Database

Col. No.	Spec. No.	Fy (ksi)	f'c (ksi)	h1 (in)	h2 (in)	t (in)	Ac (in^2)	As (in^2)	kl (in)	Pexp (k)	Pexp/Ppred by AISC 1999	AISC 2005	Eurocode	Plastic
152	HSS18	108.8	5.80	8.27	8.27	0.197	62.00	6.36	118.1	566.3	0.68	0.69	0.63	0.54
Uy, 2002														
153	CCH1	65.3	11.46	2.95	2.95	0.12	7.42	1.30	69.7	93.0	0.90	0.90	0.82	0.55
154	CCH2	58.0	11.46	2.56	2.56	0.12	5.43	1.12	69.7	66.1	0.94	0.94	0.89	0.52
155	CCM1	65.3	7.54	2.95	2.95	0.12	7.42	1.30	69.7	77.1	0.85	0.85	0.80	0.55
156	CCM2	58.0	7.54	2.56	2.56	0.12	5.43	1.12	69.7	60.4	0.96	0.96	0.94	0.57
Seo and Chung,2002														
157	C4-0	65.5	13.92	4.92	4.92	0.126	21.80	2.42	19.7	398.0	0.97	0.97	0.86	0.86
158	C8-0	65.5	13.92	4.92	4.92	0.126	21.80	2.42	39.4	427.3	1.08	1.09	0.96	0.93
159	C12-0	65.5	13.92	4.92	4.92	0.126	21.80	2.42	59.1	421.9	1.15	1.15	1.00	0.91
160	C18-0	65.5	13.92	4.92	4.92	0.126	21.80	2.42	88.6	371.0	1.18	1.19	1.00	0.80
161	C24-0	65.5	13.92	4.92	4.92	0.126	21.80	2.42	11.8	245.3	0.59	0.59	0.53	0.53
162	C30-0	65.5	13.92	4.92	4.92	0.126	21.80	2.42	147.6	169.5	0.88	0.90	0.78	0.37
Kang et al., 2001														
163	KOM2001	46.1	3.59	7.87	7.87	0.126	58.10	3.90	23.6	354.7	1.00	1.00	0.91	0.91
164	KOM2001	46.1	3.59	9.84	9.84	0.126	91.98	4.90	29.5	477.3	0.95	0.95	0.86	0.86
165	KOM2001	46.1	3.59	11.81	11.81	0.126	133.61	5.89	35.4	618.2	0.91	0.91	0.82	0.82
166	KOM2001	52.8	3.59	7.87	7.87	0.354	51.34	10.66	23.6	806.6	1.13	1.13	1.08	1.08
167	KOM2001	52.6	3.59	7.87	7.87	0.472	48.01	13.99	23.6	1015.5	1.15	1.15	1.12	1.12
168	KOM2001	46.1	4.40	7.87	7.87	0.126	58.10	3.90	23.6	553.7	1.40	1.40	1.27	1.27
169	KOM2001	46.1	4.40	11.81	11.81	0.126	133.61	5.89	35.4	1032.0	1.34	1.34	1.20	1.20
170	KOM2001	52.8	4.40	7.87	7.87	0.354	51.34	10.66	23.6	937.4	1.25	1.25	1.19	1.19
171	KOM2001	52.6	4.40	7.87	7.87	0.472	48.01	13.99	23.6	1115.8	1.22	1.22	1.18	1.18
Yang and Seo, 1998														
172		49.6	2.61	3.94	3.94	0.087	14.04	1.37	11.8	101.2	1.02	1.02	0.97	0.97
173		49.6	6.53	3.94	3.94	0.087	14.04	1.37	11.8	140.2	0.97	0.97	0.88	0.88

Table A-5 - RCFT Column Database

Col. No.	Spec. No.	Fy (ksi)	f'c (ksi)	h1 (in)	h2 (in)	t (in)	Ac (in^2)	As (in^2)	kl (in)	Pexp (k)	Pexp/Ppred by AISC 1999	AISC 2005	Eurocode	Plastic
174		56.1	2.61	3.94	3.94	0.116	13.68	1.77	11.8	141.4	1.10	1.10	1.05	1.05
175		56.1	6.53	3.94	3.94	0.116	13.68	1.77	11.8	178.7	1.03	1.03	0.95	0.95
176		52.9	2.61	3.94	3.94	0.157	13.10	2.32	11.8	181.1	1.20	1.20	1.15	1.15
177		52.9	6.53	3.94	3.94	0.157	13.10	2.32	11.8	217.2	1.12	1.12	1.04	1.04
Kang et al., 2002														
178	KLM2002	49.7	7.86	1.97	1.97	0.114	13.10	2.32	5.9	70.1	0.35	0.35	0.32	0.32
179	KLM2002	40.6	7.86	2.95	2.95	0.126	13.10	2.32	8.9	146.9	0.82	0.82	0.75	0.75
180	KLM2002	40.2	7.86	3.94	3.94	0.126	13.10	2.32	11.8	204.5	1.14	1.14	1.04	1.04
181	KLM2002	52.9	7.86	3.94	3.94	0.091	13.10	2.32	11.8	166.7	0.80	0.80	0.74	0.74
182	KLM2002	49.7	6.77	1.97	1.97	0.114	13.10	2.32	5.9	60.2	0.32	0.32	0.30	0.30
183	KLM2002	40.6	6.77	2.95	2.95	0.126	13.10	2.32	8.9	121.7	0.72	0.72	0.67	0.67
184	KLM2002	40.2	6.77	3.94	3.94	0.126	13.10	2.32	11.8	166.8	1.00	0.99	0.92	0.92
185	KLM2002	52.9	6.77	3.94	3.94	0.091	13.10	2.32	11.8	129.6	0.66	0.66	0.61	0.61
Lee et al., 2002														
186	LPK2002	51.4	8.03	2.95	2.95	0.126	13.10	2.32	10.8	168.8	0.82	0.82	0.75	0.75
187	LPK2002	51.4	8.03	2.95	2.95	0.126	13.10	2.32	21.6	140.5	0.68	0.68	0.62	0.60
188	LPK2002	51.4	8.03	2.95	2.95	0.126	13.10	2.32	32.4	132.9	0.69	0.69	0.62	0.57
189	LPK2002	53.9	8.03	3.94	3.94	0.126	13.10	2.32	14.7	234.5	1.11	1.10	1.02	1.02
190	LPK2002	53.9	8.03	3.94	3.94	0.126	13.10	2.32	29.5	198.5	0.96	0.96	0.88	0.86
191	LPK2002	53.9	8.03	3.94	3.94	0.126	13.10	2.32	44.2	189.3	0.96	0.96	0.88	0.82
192	LPK2002	54.3	8.03	3.94	3.94	0.091	13.10	2.32	15.0	195.9	0.92	0.92	0.85	0.85
193	LPK2002	54.3	8.03	3.94	3.94	0.091	13.10	2.32	30.0	175.9	0.87	0.86	0.79	0.76
194	LPK2002	54.3	8.03	3.94	3.94	0.091	13.10	2.32	45.1	158.8	0.84	0.82	0.75	0.69
195	LPK2002	51.4	8.03	2.95	2.95	0.126	13.10	2.32	10.8	158.7	0.77	0.77	0.71	0.71
196	LPK2002	51.4	8.03	2.95	2.95	0.126	13.10	2.32	21.6	140.4	0.68	0.68	0.62	0.60
197	LPK2002	51.4	8.03	2.95	2.95	0.126	13.10	2.32	32.4	113.7	0.59	0.59	0.53	0.49
198	LPK2002	53.9	8.03	3.94	3.94	0.126	13.10	2.32	14.7	229.3	1.08	1.08	1.00	1.00
199	LPK2002	53.9	8.03	3.94	3.94	0.126	13.10	2.32	29.5	197.3	0.96	0.95	0.88	0.86

Table A-5 - RCFT Column Database

Col. No.	Spec. No.	Fy (ksi)	f'c (ksi)	h1 (in)	h2 (in)	t (in)	Ac (in^2)	As (in^2)	kl (in)	Pexp (k)	Pexp/Ppred by AISC 1999	AISC 2005	Eurocode	Plastic
200	LPK2002	53.9	8.03	3.94	3.94	0.126	13.10	2.32	44.2	179.4	0.91	0.91	0.83	0.78
201	LPK2002	54.3	8.03	3.94	3.94	0.091	13.10	2.32	15.0	196.4	0.93	0.92	0.85	0.85
202	LPK2002	54.3	8.03	3.94	3.94	0.091	13.10	2.32	30.0	161.2	0.79	0.78	0.72	0.70
203	LPK2002	54.3	8.03	3.94	3.94	0.091	13.10	2.32	45.1	151.6	0.80	0.78	0.71	0.66
204	LPK2002	51.4	8.03	2.95	2.95	0.126	13.10	2.32	10.8	171.3	0.83	0.83	0.76	0.76
205	LPK2002	51.4	8.03	2.95	2.95	0.126	13.10	2.32	21.6	146.4	0.71	0.71	0.65	0.63
206	LPK2002	51.4	8.03	2.95	2.95	0.126	13.10	2.32	32.4	138.1	0.71	0.71	0.65	0.59
207	LPK2002	53.9	8.03	3.94	3.94	0.126	13.10	2.32	14.7	239.0	1.13	1.13	1.04	1.04
208	LPK2002	53.9	8.03	3.94	3.94	0.126	13.10	2.32	29.5	206.3	1.00	1.00	0.92	0.90
209	LPK2002	53.9	8.03	3.94	3.94	0.126	13.10	2.32	44.2	195.2	0.99	0.99	0.90	0.85
210	LPK2002	54.3	8.03	3.94	3.94	0.091	13.10	2.32	15.0	210.8	0.99	0.99	0.91	0.91
211	LPK2002	54.3	8.03	3.94	3.94	0.091	13.10	2.32	30.0	180.8	0.89	0.88	0.81	0.78
212	LPK2002	54.3	8.03	3.94	3.94	0.091	13.10	2.32	45.1	167.3	0.88	0.86	0.79	0.72
Seo et al., 2002														
213	SKA2002	65.7	9.99	4.92	4.92	0.117	21.93	2.25	19.7	337.8	1.02	1.02	0.92	0.92
214	SKA2002	63.2	9.99	4.93	4.93	0.117	22.01	2.25	19.7	337.1	1.04	1.04	0.93	0.93
215	SKA2002	64.5	9.29	4.92	4.92	0.117	21.97	2.25	39.4	341.1	1.12	1.12	1.01	0.98
216	SKA2002	65.7	9.79	4.92	4.92	0.117	21.93	2.25	59.1	307.9	1.04	1.04	0.92	0.85
217	SKA2002	63.2	9.79	4.93	4.93	0.117	22.01	2.25	59.1	320.0	1.10	1.10	0.97	0.89
218	SKA2002	65.7	9.02	4.92	4.92	0.117	21.93	2.25	88.6	296.2	1.19	1.20	1.05	0.86
219	SKA2002	63.2	9.02	4.93	4.93	0.117	22.01	2.25	88.6	295.1	1.20	1.21	1.06	0.87
220	SKA2002	65.7	9.05	4.92	4.92	0.117	21.93	2.25	88.6	233.5	0.94	0.95	0.83	0.67
221	SKA2002	65.7	8.89	4.92	4.92	0.117	21.93	2.25	118.1	182.0	0.89	0.90	0.82	0.53
222	SKA2002	63.2	8.89	4.93	4.93	0.117	22.01	2.25	118.1	201.1	0.99	1.00	0.91	0.60

Table A-6 - RCFT Beam-Column Database

Col. No.	Spec. No.	Fy ksi	f'c ksi	h1 in	h2 in	ts in	Ac in^2	As in^2	kl in	Pexp k	ex in	ey in	Pexp/Ppred by AISC 1999	AISC 2005	Eurocode 4	Plastic
Furlong, 1967																
1		70.3	6.50	5.00	5.00	0.189	21.36	3.64	36.0	250.0	1.24		1.32	1.13	1.09	0.98
2		70.3	6.50	5.00	5.00	0.189	21.36	3.64	36.0	150.0	2.43		1.14	0.93	0.96	0.81
3		70.3	6.50	5.00	5.00	0.189	21.36	3.64	36.0	150.0	2.87		1.27	1.02	1.08	0.90
4		70.3	6.50	5.00	5.00	0.189	21.36	3.64	36.0	100.0	1.50		0.58	0.49	0.47	0.42
5		48.0	3.40	4.00	4.00	0.084	14.67	1.33	36.0	84.0	0.53		1.27	1.09	1.04	0.95
6		48.0	3.40	4.00	4.00	0.084	14.67	1.33	36.0	84.0	0.53		1.28	1.09	1.04	0.95
7		48.0	3.40	4.00	4.00	0.084	14.67	1.33	36.0	54.4	1.69		1.44	1.09	1.13	0.94
8		48.0	3.40	4.00	4.00	0.084	14.67	1.33	36.0	20.2	5.19		1.22	0.83	1.16	0.91
9		48.0	3.40	4.00	4.00	0.084	14.67	1.33	36.0	20.1	5.68		1.31	0.89	1.27	1.00
10		48.0	4.18	4.00	4.00	0.125	14.05	1.95	36.0	98.4	1.21		1.53	1.26	1.23	1.07
11		48.0	4.18	4.00	4.00	0.125	14.05	1.95	36.0	68.8	2.35		1.60	1.24	1.35	1.10
12		48.0	4.18	4.00	4.00	0.125	14.05	1.95	36.0	67.8	2.39		1.59	1.23	1.35	1.09
13		48.0	4.18	4.00	4.00	0.125	14.05	1.95	36.0	58.6	3.24		1.70	1.29	1.52	1.22
14		48.0	4.18	4.00	4.00	0.125	14.05	1.95	36.0	29.0	7.21		1.60	1.16	1.64	1.31
15		48.0	4.18	4.00	4.00	0.125	14.05	1.95	36.0	28.8	6.70		1.49	1.09	1.52	1.20
16		48.0	4.18	4.00	4.00	0.125	14.05	1.95	36.0	9.0	18.33		1.16	0.82	1.32	1.06
Knowles and Park, 1969																
17		47.0	6.00	3.00	3.00	0.131	7.48	1.50	32.0	77.8	0.30		1.06	0.95	0.91	0.82
18		47.0	6.00	3.00	3.00	0.131	7.48	1.50	56.0	63.2	0.30		1.00	0.85	0.85	0.67
19		47.0	6.00	3.00	3.00	0.131	7.48	1.50	32.0	48.7	1.00		1.08	0.87	0.88	0.73
20		47.0	6.00	3.00	3.00	0.131	7.48	1.50	56.0	35.2	1.00		0.88	0.67	0.74	0.53
Bridge, 1976																
21		42.2	4.38	7.87	7.87	0.394	50.22	11.78	83.9	439.7	1.50		1.13	1.00	0.98	0.87
22		36.8	5.08	5.91	5.91	0.256	29.09	5.78	120.1	152.9	1.50		1.09	0.84	0.91	0.66
23		36.8	5.08	5.91	5.91	0.256	29.09	5.78	120.1	115.3	2.52		1.07	0.79	0.92	0.64

Table A-6 - RCFT Beam-Column Database

Col. No.	Spec. No.	Fy ksi	f'c ksi	h1 in	h2 in	ts in	Ac in^2	As in^2	kl in	Pexp k	ex in	ey in	Pexp/Ppred by AISC 1999	AISC 2005	Eurocode 4	Plastic
Shakir-Khalil and Zeghiche, 1989																
24	2	56.0	5.22	3.15	4.72	0.197	11.93	2.92	126.4	88.3	0.94		1.11	0.87	1.08	0.57
25	3	55.8	5.22	3.15	4.72	0.197	11.93	2.90	126.4	52.2	2.36		0.96	0.72	0.96	0.50
26	4	55.8	5.74	4.72	3.15	0.197	11.93	2.90	115.7	58.4		0.63	0.90	0.69	0.96	0.36
27	5	49.8	5.61	4.72	3.15	0.197	11.93	2.90	115.7	47.2		1.57	1.05	0.76	1.13	0.46
Shakir-Khalil and Mouli, 1990																
28		51.8	5.18	4.72	3.15	0.197	11.25	2.88	115.7	91.5	0.31		1.25	1.02	1.32	0.53
Grauers, 1993																
29	1	44.1	6.82	4.72	4.72	0.197	18.76	3.57	125.8	137.1	0.79		1.31	0.97	1.17	0.66
30	2	63.5	6.67	4.72	4.72	0.197	18.76	3.57	125.8	157.4	0.79		1.25	0.96	1.19	0.61
31	3	47.4	13.92	4.72	4.72	0.197	18.76	3.57	125.8	159.6	0.79		1.29	0.84	1.09	0.50
32	4	63.7	13.92	4.72	4.72	0.197	18.76	3.57	125.8	186.6	0.79		1.32	0.91	1.18	0.52
33	5	46.8	5.66	4.72	4.72	0.315	16.76	5.56	125.8	166.4	0.79		1.14	0.93	1.07	0.64
34	6	43.5	6.67	4.72	4.72	0.315	16.76	5.56	125.8	173.1	0.79		1.21	0.97	1.12	0.67
35	7	54.5	6.82	4.72	4.72	0.315	16.76	5.56	125.8	195.6	0.79		1.21	0.97	1.16	0.65
36	8	46.8	14.94	4.72	4.72	0.315	16.76	5.56	125.8	184.4	0.79		1.11	0.80	1.00	0.49
37	9	55.0	14.94	4.72	4.72	0.315	16.76	5.56	125.8	224.8	0.79		1.26	0.92	1.16	0.55
38	10	55.0	5.66	4.72	4.72	0.315	16.76	5.56	125.8	184.4	0.79		1.16	0.95	1.12	0.63
39	11	54.5	13.49	4.72	4.72	0.315	16.76	5.56	125.8	231.6	0.79		1.32	0.98	1.22	0.60
40	12	52.8	13.49	4.72	4.72	0.315	16.76	5.56	125.8	215.8	0.79		1.25	0.92	1.15	0.57
41	13	52.8	11.60	4.72	4.72	0.315	16.76	5.56	125.8	260.8	0.39		1.28	1.04	1.18	0.63
42	14	55.0	11.60	4.72	4.72	0.315	16.76	5.56	125.8	362.0	0.79		2.10	1.59	1.94	0.99
43	16	57.4	13.92	4.72	4.72	0.315	16.76	5.56	129.0	233.8	0.79		1.33	0.98	1.24	0.58
44	17	58.6	13.34	4.72	4.72	0.315	16.76	5.56	125.8	227.1	0.79		1.26	0.94	1.17	0.57
45	18	58.6	13.34	4.72	4.72	0.315	16.76	5.56	125.8	168.6	0.79		0.93	0.70	0.87	0.42
46	25	57.4	13.34	4.72	4.72	0.315	16.76	5.56	125.8	215.8	0.79		1.21	0.90	1.12	0.55
47	23	55.0	4.50	4.72	4.72	0.315	16.76	5.56	66.8	242.8	0.79		1.14	1.04	1.06	0.88
48	24	55.0	13.34	4.72	4.72	0.315	16.76	5.56	66.8	292.3	0.79		1.16	0.94	0.93	0.76

Table A-6 - RCFT Beam-Column Database

Col. No.	Spec. No.	Fy ksi	f'c ksi	h1 in	h2 in	ts in	Ac in^2	As in^2	kl in	Pexp k	ex in	ey in	Pexp/Ppred by AISC 1999	AISC 2005	Eurocode 4	Plastic
49	27	55.0	4.79	9.84	9.84	0.315	84.87	12.00	125.8	764.4	0.79		1.10	0.99	0.96	0.84
50	28	55.0	13.20	9.84	9.84	0.315	84.87	12.00	125.8	1191.5	0.79		1.27	0.98	0.91	0.78
Baba, Fujimoto, Mukai, and Nishiyama, 1995																
51	ER4-A-4-4.5	37.9	5.95	5.87	5.87	0.172	30.48	3.93	17.6	175.0	0.18		0.67	0.62	0.57	0.56
52	ER4-A-4-20	37.9	5.95	5.87	5.87	0.172	30.48	3.93	17.6	59.4	0.79		0.33	0.26	0.24	0.23
53	ER4-C-2-6	37.9	3.68	8.50	8.50	0.172	66.57	5.75	25.5	259.0	0.24		0.69	0.65	0.60	0.59
54	ER4-C-2-20	37.9	3.68	8.50	8.50	0.172	66.57	5.75	25.5	115.6	0.79		0.39	0.34	0.31	0.30
55	ER4-C-4-6	37.9	5.95	8.50	8.50	0.172	66.57	5.75	25.5	313.0	0.24		0.66	0.61	0.55	0.54
56	ER4-C-4-10	37.9	5.95	8.50	8.50	0.172	66.57	5.75	25.5	233.2	0.39		0.54	0.47	0.43	0.42
57	ER4-C-4-20	37.9	5.95	8.50	8.50	0.172	66.57	5.75	25.5	128.9	0.79		0.37	0.29	0.26	0.25
58	ER4-C-8-6	37.9	11.62	8.50	8.50	0.172	66.57	5.75	25.5	456.2	0.24		0.67	0.56	0.49	0.49
59	ER4-C-8-10	37.9	11.62	8.50	8.50	0.172	66.57	5.75	25.5	327.2	0.39		0.55	0.42	0.37	0.36
60	ER4-D-4-6	37.9	5.95	12.76	12.76	0.172	154.03	8.68	38.3	747.0	0.24		0.78	0.71	0.63	0.62
61	ER4-D-4-20	37.9	5.95	12.76	12.76	0.172	154.03	8.68	38.3	336.2	0.79		0.46	0.35	0.31	0.30
62	ER6-A-4-4.5	89.4	5.95	5.67	5.67	0.250	26.71	5.43	17.0	380.1	0.18		0.68	0.67	0.64	0.63
63	ER6-A-4-20	89.4	5.95	5.67	5.67	0.250	26.71	5.43	17.0	141.7	0.79		0.33	0.32	0.30	0.29
64	ER6-C-2-6	89.4	3.68	8.27	8.27	0.250	60.33	8.03	24.8	545.1	0.24		0.66	0.65	0.63	0.62
65	ER6-C-2-025	89.4	3.68	8.27	8.27	0.250	60.33	8.03	17.4	235.0	0.79		0.34	0.33	0.31	0.30
66	ER6-C-4-6	89.4	5.95	8.27	8.27	0.250	60.33	8.03	24.8	613.8	0.24		0.67	0.65	0.61	0.61
67	ER6-C-4-10	89.4	5.95	8.27	8.27	0.250	60.33	8.03	24.8	473.8	0.39		0.54	0.52	0.50	0.49
68	ER6-C-4-30	89.4	5.95	8.27	8.27	0.250	60.33	8.03	24.8	193.8	1.18		0.29	0.26	0.25	0.24
69	ER6-C-8-6	89.4	11.62	8.27	8.27	0.250	60.33	8.03	24.8	766.4	0.24		0.66	0.63	0.58	0.57
70	ER6-C-8-20	89.4	11.62	8.27	8.27	0.250	60.33	8.03	24.8	340.6	0.79		0.37	0.32	0.30	0.29
71	ER6-D-4-10	89.4	5.95	12.52	12.52	0.250	144.45	12.29	37.6	939.8	0.39		0.58	0.56	0.52	0.51
72	ER6-D-4-30	89.4	5.95	12.52	12.52	0.250	144.45	12.29	37.6	450.3	1.18		0.34	0.31	0.29	0.28
73	ER8-A-4-01	120.8	5.86	4.72	4.72	0.255	17.77	4.55	9.9	52.3	1.45		0.16	0.15	0.15	0.14
74	ER8-C-2-04	120.8	3.68	6.85	6.85	0.255	40.21	6.72	14.4	374.2	0.40		0.47	0.46	0.45	0.44
75	ER8-C-2-06	120.8	3.68	6.85	6.85	0.255	40.21	6.72	14.4	566.1	0.21		0.66	0.65	0.64	0.63
76	ER8-C-4-025	120.8	5.86	6.85	6.85	0.255	40.21	6.72	14.4	261.9	0.60		0.33	0.32	0.31	0.30

Table A-6 - RCFT Beam-Column Database

Col.	Spec.	Fy	f'c	h1	h2	ts	Ac	As	kl	Pexp	ex	ey	Pexp/Ppred by			
No.	No.	ksi	ksi	in	in	in	in^2	in^2	in	k	in	in	AISC 1999	AISC 2005	Eurocode 4	Plastic
77	ER8-C-4-04	120.8	5.86	6.85	6.85	0.255	40.21	6.72	14.4	419.0	0.37		0.48	0.48	0.46	0.45
78	ER8-C-4-06	120.8	5.86	6.85	6.85	0.255	40.21	6.72	14.4	618.0	0.18		0.66	0.66	0.63	0.62
79	ER8-C-8-04	120.8	11.14	6.85	6.85	0.255	40.21	6.72	14.4	503.9	0.34		0.50	0.48	0.45	0.44
80	ER8-C-8-06	120.8	11.14	6.85	6.85	0.255	40.21	6.72	14.4	755.9	0.18		0.70	0.68	0.64	0.63
81	ER8-D-4-04	120.8	5.86	10.39	10.39	0.255	97.70	10.33	21.8	746.4	0.44		0.49	0.48	0.46	0.45
82	ER8-D-4-06	120.8	5.86	10.39	10.39	0.255	97.70	10.33	21.8	1110.5	0.21		0.69	0.68	0.64	0.64
Wang and Moore, 1997																
83	RHS7	53.7	7.25	3.15	4.72	0.248	11.24	3.67	126.1	117.0	2.17		1.73	1.28	1.71	0.89
84	RHS8	53.7	7.25	3.15	4.72	0.248	11.24	3.67	126.1	108.0	2.17		1.60	1.18	1.58	0.82
85	RHS1	53.7	7.25	4.72	3.15	0.248	11.24	3.67	157.6	82.8		2.17	1.46	1.04	1.54	0.62
86	RHS2	53.7	7.25	4.72	3.15	0.248	11.24	3.67	157.6	55.4		2.17	0.97	0.68	1.03	0.41
Matsui et al.,1997																
87	C4-1	59.7	5.93	5.90	5.90	0.168	30.98	3.86	23.6	266.4		0.99	1.17	0.99	0.93	0.88
88	C4-3	59.7	5.93	5.90	5.90	0.168	30.98	3.86	23.6	165.2		2.96	1.31	0.98	1.04	0.88
89	C4-5	59.7	5.93	5.90	5.90	0.168	30.98	3.86	23.6	115.7		4.93	1.33	0.94	1.14	0.94
90	C8-1	59.7	5.93	5.90	5.90	0.168	30.98	3.86	47.3	254.9		0.99	1.16	0.97	0.92	0.84
91	C8-3	59.7	5.93	5.90	5.90	0.168	30.98	3.86	47.3	149.6		2.96	1.22	0.90	0.98	0.79
92	C8-5	59.7	5.93	5.90	5.90	0.168	30.98	3.86	47.3	108.9		4.93	1.28	0.90	1.10	0.89
93	C12-1	59.7	5.93	5.90	5.90	0.168	30.98	3.86	70.9	230.6		0.99	1.11	0.91	0.89	0.76
94	C12-3	59.7	5.93	5.90	5.90	0.168	30.98	3.86	70.9	142.0		2.96	1.21	0.87	0.99	0.75
95	C12-5	59.7	5.93	5.90	5.90	0.168	30.98	3.86	70.9	100.1		4.93	1.19	0.84	1.06	0.82
96	C18-1	59.7	5.93	5.90	5.90	0.168	30.98	3.86	106.4	190.4		0.99	0.83	0.82	0.85	0.63
97	C18-3	59.7	5.93	5.90	5.90	0.168	30.98	3.86	106.4	124.4		2.96	1.07	0.81	1.00	0.66
98	C18-5	59.7	5.93	5.90	5.90	0.168	30.98	3.86	106.4	200.9		4.93	2.54	1.75	2.37	1.64
99	C24-1	59.7	5.93	5.90	5.90	0.168	30.98	3.86	141.8	158.6		0.99	0.84	0.77	0.90	0.52
100	C24-3	59.7	5.93	5.90	5.90	0.168	30.98	3.86	141.8	99.2		2.96	0.95	0.70	0.96	0.53
101	C24-5	59.7	5.93	5.90	5.90	0.168	30.98	3.86	141.8	73.4		4.93	1.01	0.68	1.01	0.60
102	C30-1	59.7	5.93	5.90	5.90	0.168	30.98	3.86	177.3	132.3		0.99	0.86	0.75	1.02	0.43

Table A-6 - RCFT Beam-Column Database

Col. No.	Spec. No.	Fy ksi	f'c ksi	h1 in	h2 in	ts in	Ac in^2	As in^2	kl in	Pexp k	ex in	ey in	Pexp/Ppred by AISC 1999	AISC 2005	Eurocode 4	Plastic
103	C30-3	59.7	5.93	5.90	5.90	0.168	30.98	3.86	177.3	83.9		2.96	0.92	0.66	1.02	0.45
Hardika and Gardner, 2004																
104	SNL-1	56.6	6.44	7.99	7.99	0.174	58.44	5.43	70.9	881.2	1.41		2.22	2.10	2.00	1.77
105	SNL-2	56.6	6.44	7.99	7.99	0.174	58.44	5.43	70.9	1241.3	1.02		2.58	2.71	2.54	2.31
106	SNL-3	56.6	6.44	7.99	7.99	0.174	58.44	5.43	70.9	2503.9	0.46		3.58	4.78	4.42	4.11
107	SCL-1	57.0	6.44	7.99	7.99	0.354	53.07	10.80	70.9	1734.5	1.31		2.42	2.84	2.75	2.47
108	SCL-2	57.0	6.44	7.99	7.99	0.354	53.07	10.80	70.9	3126.0	0.75		3.31	4.47	4.29	3.94
109	SCL-3	57.0	6.44	7.99	7.99	0.354	53.07	10.80	70.9	5984.8	0.37		4.95	7.74	7.34	6.89
110	SNH-1	59.7	14.30	7.99	7.99	0.354	53.07	10.80	70.9	1367.3	1.04		1.43	1.48	1.39	1.25
111	SNH-2	59.7	14.37	7.99	7.99	0.354	53.07	10.80	70.9	2288.3	0.73		1.99	2.30	2.14	1.95
112	SNH-3	59.7	14.34	7.99	7.99	0.354	53.07	10.80	70.9	1896.2	0.89		1.81	1.98	1.85	1.67
113	SCH-1	54.8	12.02	7.99	7.99	0.354	53.07	10.80	70.9	2101.9	1.12		2.51	2.62	2.47	2.23
114	SCH-2	54.8	12.60	7.99	7.99	0.354	53.07	10.80	70.9	5753.7	0.42		4.31	5.91	5.46	5.08
115	SCH-3	54.8	12.98	7.99	7.99	0.354	53.07	10.80	70.9	4822.1	0.50		3.83	4.99	4.62	4.28
Han and Yao, 2002																
116	M-4-1	49.3	3.35	7.68	5.12	0.10	36.67	2.63	30.7	196.0	0.55		0.86	1.07	1.00	0.95
117	M-4-2	49.3	3.35	7.68	5.12	0.10	36.67	2.63	30.7	182.5	0.55		0.80	0.99	0.93	0.89
118	H-4-1	49.3	3.35	7.68	5.12	0.10	36.67	2.63	30.7	164.5	0.55		0.72	0.89	0.84	0.80
119	H-4-2	49.3	3.35	7.68	5.12	0.10	36.67	2.63	30.7	166.3	0.55		0.73	0.90	0.85	0.81
120	M-5-1	49.3	3.35	7.68	5.12	0.10	36.67	2.63	30.7	145.2	1.22		1.01	0.98	0.94	0.86
121	M-5-2	49.3	3.35	7.68	5.12	0.10	36.67	2.63	30.7	137.1	1.22		0.96	0.93	0.89	0.82
122	H-5-1	49.3	3.35	7.68	5.12	0.10	36.67	2.63	30.7	112.4	1.22		0.78	0.76	0.73	0.67
123	H-5-2	49.3	3.35	7.68	5.12	0.10	36.67	2.63	30.7	115.5	1.22		0.81	0.78	0.75	0.69
124	M-7-1	49.3	3.35	7.68	5.12	0.10	36.67	2.63	92.1	150.6	0.55		0.81	0.92	0.92	0.73
125	M-7-2	49.3	3.35	7.68	5.12	0.10	36.67	2.63	92.1	142.7	0.55		0.77	0.87	0.87	0.69
126	H-7-1	49.3	3.35	7.68	5.12	0.10	36.67	2.63	92.1	118.0	0.55		0.64	0.72	0.72	0.57
127	H-7-2	49.3	3.35	7.68	5.12	0.10	36.67	2.63	92.1	112.4	0.55		0.61	0.68	0.69	0.55

Table A-6 - RCFT Beam-Column Database

Col. No.	Spec. No.	Fy ksi	f'c ksi	h1 in	h2 in	ts in	Ac in^2	As in^2	kl in	Pexp k	ex in	ey in	Pexp/Ppred by AISC 1999	AISC 2005	Eurocode 4	Plastic
Uy, 2000																
128	HSS3	108.8	4.35	4.33	4.33	0.197	15.50	3.26	118.1	349.4	0.59		2.11	2.06	2.73	1.08
129	HSS4	108.8	4.35	4.33	4.33	0.197	15.50	3.26	118.1	287.9	1.18		2.14	1.96	2.73	1.10
130	HSS7	108.8	4.35	6.30	6.30	0.197	34.88	4.81	118.1	293.9	1.57		0.93	0.92	1.06	0.68
131	HSS10	108.8	4.35	6.30	6.30	0.197	34.88	4.81	118.1	454.8	0.98		1.20	1.09	1.01	0.94
132	HSS11	108.8	4.35	6.30	6.30	0.197	34.88	4.81	118.1	444.7	1.97		1.52	0.94	1.11	0.93
133	HSS16	108.8	4.64	8.27	8.27	0.197	62.00	6.36	118.1	698.0	0.98		1.16	1.06	1.00	0.89
134	HSS17	108.8	4.64	8.27	8.27	0.197	62.00	6.36	118.1	588.1	1.97		1.41	0.89	1.07	0.87
Seo and Chung, 2002																
135	C4-1	65.5	13.92	4.92	4.92	0.126	21.80	2.42	19.7	282.5	0.81		1.16	0.96	0.89	0.83
136	C4-3	65.5	13.92	4.92	4.92	0.126	21.80	2.42	19.7	147.9	2.42		1.45	0.79	0.96	0.82
137	C8-1	65.5	13.92	4.92	4.92	0.126	21.80	2.42	39.4	266.7	0.81		1.15	0.94	0.89	0.78
138	C8-3	65.5	13.92	4.92	4.92	0.126	21.80	2.42	39.4	138.0	2.42		1.39	0.75	0.95	0.76
139	C12-1	65.5	13.92	4.92	4.92	0.126	21.80	2.42	59.1	253.9	0.81		1.19	0.94	0.94	0.74
140	C12-3	65.5	13.92	4.92	4.92	0.126	21.80	2.42	59.1	128.2	2.42		1.34	0.72	0.98	0.71
141	C18-1	65.5	13.92	4.92	4.92	0.126	21.80	2.42	88.6	186.0	0.81		1.03	0.77	0.87	0.54
142	C18-3	65.5	13.92	4.92	4.92	0.126	21.80	2.42	88.6	100.3	2.42		1.15	0.61	0.94	0.55
143	C24-1	65.5	13.92	4.92	4.92	0.126	21.80	2.42	118.1	149.8	0.81		1.02	0.74	0.92	0.44
144	C24-3	65.5	13.92	4.92	4.92	0.126	21.80	2.42	118.1	81.2	2.42		1.05	0.56	0.97	0.45
145	C30-3	65.5	13.92	4.92	4.92	0.126	21.80	2.42	147.6	64.6	2.42		0.95	0.53	0.95	0.36
Seo et al., 2002																
146	C04-1-00	64.5	9.99	4.92	4.92	0.118	21.96	2.26	19.7	253.7	0.83		1.20	1.09	1.01	0.94
147	C04-1-15	64.5	9.99	4.92	4.92	0.118	21.96	2.26	19.7	238.4	0.21		1.10	1.02	1.36	0.88
148	C04-1-30	64.5	9.99	4.92	4.92	0.118	21.96	2.26	19.7	244.9	0.41		1.05	1.01	1.36	0.88
149	C04-1-45	64.5	9.99	4.92	4.92	0.118	21.96	2.26	19.7	238.4	0.58		0.91	0.94	1.25	0.82
150	C04-3-00	64.5	9.99	4.92	4.92	0.118	21.96	2.26	19.7	137.5	2.48		1.52	0.94	1.11	0.93
151	C04-3-15	64.5	9.99	4.92	4.92	0.118	21.96	2.26	19.7	132.6	0.64		1.42	0.89	1.39	0.88
152	C04-3-30	64.5	9.99	4.92	4.92	0.118	21.96	2.26	19.7	129.7	1.24		1.27	0.82	1.24	0.79

Table A-6 - RCFT Beam-Column Database

Col. No.	Spec. No.	Fy ksi	f'c ksi	h1 in	h2 in	ts in	Ac in^2	As in^2	kl in	Pexp k	ex in	ey in	Pexp/Ppred by AISC 1999	AISC 2005	Eurocode 4	Plastic
153	C04-3-45	64.5	9.99	4.92	4.92	0.118	21.96	2.26	19.7	132.1	1.75		1.09	0.75	1.11	0.69
154	C08-1-00	64.5	9.29	4.92	4.92	0.118	21.96	2.26	39.4	230.3	0.83		1.16	1.06	1.00	0.89
155	C08-1-15	64.5	9.29	4.92	4.92	0.118	21.96	2.26	39.4	228.5	0.21		1.12	1.04	1.43	0.88
156	C08-1-30A	65.7	9.29	4.92	4.92	0.117	21.93	2.25	39.4	208.5	0.41		0.95	0.92	1.25	0.78
157	C08-1-30B	63.2	9.29	4.93	4.93	0.119	21.98	2.28	39.4	215.1	0.41		1.00	0.96	1.30	0.81
158	C08-1-45	64.5	9.29	4.92	4.92	0.118	21.96	2.26	39.4	225.6	0.58		0.92	0.95	1.28	0.81
159	C08-3-00	64.5	9.29	4.92	4.92	0.118	21.96	2.26	39.4	123.8	2.48		1.41	0.89	1.07	0.87
160	C08-3-15	64.5	9.29	4.92	4.92	0.118	21.96	2.26	39.4	120.7	0.64		1.33	0.86	1.33	0.82
161	C08-3-30	64.5	9.29	4.92	4.92	0.118	21.96	2.26	39.4	123.1	1.24		1.24	0.82	1.24	0.77
162	C08-3-45	64.5	9.29	4.92	4.92	0.118	21.96	2.26	39.4	121.1	1.75		1.04	0.74	1.08	0.65
163	C12-0-00	64.5	9.79	4.92	4.92	0.118	21.96	2.26	59.1	198.2	0.83		1.07	0.93	0.91	0.74
164	C12-1-15	64.5	9.79	4.92	4.92	0.118	21.96	2.26	59.1	206.1	0.21		1.09	0.95	1.32	0.77
165	C12-1-30	64.5	9.79	4.92	4.92	0.118	21.96	2.26	59.1	200.7	0.41		0.99	0.90	1.24	0.73
166	C12-1-45	64.5	9.79	4.92	4.92	0.118	21.96	2.26	59.1	200.9	0.58		0.89	0.86	1.16	0.70
167	C12-3-00	64.5	9.79	4.92	4.92	0.118	21.96	2.26	59.1	109.4	2.48		1.29	0.79	0.99	0.75
168	C12-3-15	64.5	9.79	4.92	4.92	0.118	21.96	2.26	59.1	111.7	0.64		1.28	0.79	1.28	0.75
169	C12-3-30	64.5	9.79	4.92	4.92	0.118	21.96	2.26	59.1	109.9	1.24		1.16	0.74	1.15	0.67
170	C12-3-45	64.5	9.79	4.92	4.92	0.118	21.96	2.26	59.1	110.8	1.75		1.00	0.68	1.00	0.59
171	C18-1-00	64.5	9.79	4.92	4.92	0.118	21.96	2.26	88.6	178.0	0.83		1.12	0.92	1.01	0.67
172	C18-1-15	64.5	9.79	4.92	4.92	0.118	21.96	2.26	88.6	179.6	0.21		1.11	0.92	1.27	0.67
173	C18-1-30	64.5	9.79	4.92	4.92	0.118	21.96	2.26	88.6	180.7	0.41		1.06	0.91	1.23	0.65
174	C18-1-45	64.5	9.79	4.92	4.92	0.118	21.96	2.26	88.6	176.4	0.58		0.93	0.85	1.13	0.61
175	C18-3-0	64.5	9.79	4.92	4.92	0.118	21.96	2.26	88.6	98.4	2.48		1.27	0.76	1.07	0.67
176	C18-3-15	64.5	9.79	4.92	4.92	0.118	21.96	2.26	88.6	101.6	0.64		1.28	0.77	1.29	0.68
177	C18-3-30	64.5	9.79	4.92	4.92	0.118	21.96	2.26	88.6	95.3	1.24		1.10	0.69	1.12	0.59
178	C18-3-45	64.5	9.79	4.92	4.92	0.118	21.96	2.26	88.6	97.8	1.75		0.98	0.65	0.99	0.52
179	C24--1-00	64.5	9.79	4.92	4.92	0.118	21.96	2.26	118.1	135.3	0.83		1.04	0.82	1.03	0.51
180	C24-1-15	64.5	9.79	4.92	4.92	0.118	21.96	2.26	118.1	133.9	0.21		1.01	0.80	1.11	0.50
181	C24-1-30	64.5	9.79	4.92	4.92	0.118	21.96	2.26	118.1	131.0	0.41		0.94	0.77	1.05	0.47
182	C24-1-45	64.5	9.79	4.92	4.92	0.118	21.96	2.26	118.1	128.3	0.58		0.85	0.73	0.97	0.45

Table A-6 - RCFT Beam-Column Database

Col. No.	Spec. No.	Fy ksi	f'c ksi	h1 in	h2 in	ts in	Ac in^2	As in^2	kl in	Pexp k	ex in	ey in	Pexp/Ppred by AISC 1999	AISC 2005	Eurocode 4	Plastic
183	C24-3-00	64.5	9.79	4.92	4.92	0.118	21.96	2.26	118.1	75.7	2.48		1.09	0.65	1.05	0.52
184	C24-3-15	64.5	9.79	4.92	4.92	0.118	21.96	2.26	118.1	75.7	0.64		1.07	0.64	1.11	0.51
185	C24-3-30	64.5	9.79	4.92	4.92	0.118	21.96	2.26	118.1	74.8	1.24		0.98	0.60	1.02	0.46
186	C24-3-45	64.5	9.79	4.92	4.92	0.118	21.96	2.26	118.1	74.4	1.75		0.86	0.55	0.90	0.39
187	C30-1-00	64.5	9.79	4.92	4.92	0.118	21.96	2.26	147.6	110.6	0.83		1.05	0.82	1.10	0.42
188	C-30-1-15	64.5	9.79	4.92	4.92	0.118	21.96	2.26	147.6	111.5	0.21		1.05	0.82	1.16	0.42
189	C30-1-30	64.5	9.79	4.92	4.92	0.118	21.96	2.26	147.6	110.6	0.41		1.00	0.80	1.10	0.40
190	C30-1-45	64.5	9.79	4.92	4.92	0.118	21.96	2.26	147.6	111.5	0.58		0.94	0.78	1.04	0.39
191	C30-3-00	64.5	9.79	4.92	4.92	0.118	21.96	2.26	147.6	67.0	2.48		1.10	0.67	1.15	0.46
192	C30-3-15	64.5	9.79	4.92	4.92	0.118	21.96	2.26	147.6	65.6	0.64		1.06	0.64	1.15	0.44
193	C30-3-30	64.5	9.79	4.92	4.92	0.118	21.96	2.26	147.6	65.2	1.24		0.99	0.61	1.07	0.40
194	C30-3-45	64.5	9.79	4.92	4.92	0.118	21.96	2.26	147.6	64.3	1.75		0.87	0.57	0.95	0.34
Biaxial Bending																
Bridge, 1976																
1	B1	45.4	5.00	7.87	7.87	0.394	50.22	11.78	83.9	490.1	1.30	0.75				
2	B2	46.0	4.80	7.87	7.87	0.394	50.22	11.78	83.9	486.0	1.06	1.06				
3	B3	46.3	5.48	7.87	7.87	0.394	50.22	11.78	120.1	457.9	1.30	0.75				
4	B4	46.0	4.65	7.87	7.87	0.394	50.22	11.78	120.1	364.9	1.78	1.78				
Shakir-Khalil and Zeghiche, 1989																
5	B5	49.8	5.22	3.15	4.72	0.197	11.93	2.90	126.4	60.2	0.94					
	Minor Axis Data			4.72	3.15				115.7			2.36				
6	B6	51.8	5.10	3.15	4.72	0.197	11.93	2.90	126.4	36.0	2.36					
	Minor Axis Data			4.72	3.15				115.7			1.57				
Shakir-Khalil and Mouli, 1990																
7	B7	49.4	5.63	3.15	4.72	0.197	11.93	2.90	126.4	78.2	0.47					
	Minor Axis Data			4.72	3.15				115.7			0.32				

Table A-6 - RCFT Beam-Column Database

Col. No.	Spec. No.	Fy ksi	f'c ksi	h1 in	h2 in	ts in	Ac in^2	As in^2	kl in	Pexp k	ex in	ey in	Pexp/Ppred by AISC 1999	AISC 2005	Eurocode 4	Plastic
8	B8	49.4	5.87	3.15	4.72	0.197	11.93	2.90	126.4	44.6	1.65					
	Minor Axis Data			4.72	3.15				115.7			1.10				
9	B9	52.6	5.67	3.15	4.72	0.197	11.93	2.90	126.4	4.7	0.94					
	Minor Axis Data			4.72	3.15				115.7			1.57				
10	B10	52.6	5.22	3.15	4.72	0.197	11.93	2.90	126.4	47.2	2.36					
	Minor Axis Data			4.72	3.15				115.7			0.63				
11	B11	50.3	5.55	3.94	5.91	0.197	18.60	3.50	126.4	134.0	0.59					
	Minor Axis Data			5.91	3.94				115.7			0.39				
12	B12	49.3	5.61	3.94	5.91	0.197	18.60	3.50	126.4	74.0	1.77					
	Minor Axis Data			5.91	3.94				115.7			1.18				
13	B13	49.3	5.74	3.94	5.91	0.197	18.60	3.50	126.4	57.2	2.95					
	Minor Axis Data			5.91	3.94				115.7			1.97				

APPENDIX B

DESIGN EXAMPLES

Design Example 1: Encased composite column

Overall dimension: 14 in. x 14 in.

Concrete strength: 6.25 ksi

Steel section: W8 x 40

Steel yield strength: 57.26 ksi

Modulus of steel: 29000 ksi

Steel area: 11.91 in.^{2}

I_{xx} = 146 in.^{4}

I_{yy} = 49.1 in.^{4}

r_x = 3.53 in.

r_y = 2.04 in.

Longitudinal reinforcement: 4#7 bars

Area of reinforcement (A_{sr}): 2.4 in.^{2}

Moment of inertia for reinforcement: 53.2 in.^{4}

Reinforcement yield strength: 69.95 ksi

Length of column: 68 in.

Design by AISC 1999:

From Chapter I

$$F_{my} = F_y + c_1 F_{yr}(A_r / A_s) + c_2 f_c^{'}(A_c / A_s)$$

$$= 57.26 + 0.7 \times 69.95 \times (2.4 / 11.91) + 0.6 \times 6.25 \times (181.69 / 11.91)$$

$= 124.3$

$$E_m = E + c_3 E_c (A_c / A_s)$$

$$= 29000 + 0.2 \times 4592.79 \times (181.69 / 11.91)$$

$= 43013$

r_m = max(r_x,0.3d)=max(3.53,0.3x14=4.2)

= 4.2 in.

From Chapter E:

$$\lambda_m = \frac{kl}{r_m \pi}\sqrt{\frac{F_{my}}{E_m}}$$

$$\lambda_m = \frac{68}{4.2\pi}\sqrt{\frac{124.3}{43013}}$$

=0.277

$P_u = \phi_c P_n = \phi_c A_s F_{cr} = \phi_c A_s (0.658^{\lambda^2})$

= 0.85 x 11.91 x 120.4

= 1218.9 kips

<u>Design by Proposed AISC 2005:</u>

$$C_1 = 0.1 + 2(\frac{A_s}{A_c + A_s}) \le 0.3$$

$$= 0.1 + 2(\frac{11.91}{181.69 + 11.91}) = 0.22 \le 0.3$$

$EI_{eff} = E_s I_s + 0.5\ E_r I_{sr} + C_3\ E_c I_c$

$= 29000 \text{ x } 49.1 + 0.5 \text{ x } 29000 \text{ x } 53.2 + 0.22 \text{ x } 4592.79 \text{ x } 3152.2$

$= 5{,}424{,}333$

$$P_0 = A_s F_y + A_{sr} F_{yr} + 0.85 f_c^{'}$$

$= 11.91 \text{ x } 57.26 + 2.4 \text{ x } 69.95 + 0.85 \text{ x } 6.25$

$= 1815.07$ kips

$$P_E = \frac{(EI)_e \pi^2}{(KL)^2}$$

$$= \frac{(5{,}424{,}333) \times \pi^2}{(68)^2}$$

$=11578$ kips

$$\alpha = \sqrt{P_o / P_e} = \sqrt{1815.07/11578} = 0.4$$

$$P_u = \phi_c P_n = \phi_c P_o (0.658^{\alpha^2})$$

$$= 0.75 \times 1815.07 \times (0.658^{0.4^2})$$

$=1274.85$ kips

Design by Eurocode 4:

E_{ce} = 600 x 6.25 = 3750 ksi

γ_s = 1.1
γ_c = 1.5
γ_r = 1.15

$$P_{pl} = A_s \frac{F_y}{\gamma_s} + A_c \frac{0.85 f_c'}{\gamma_c} + A_r \frac{F_{yr}}{\gamma_r}$$

= 11.91 x 57.26 / 1.1+181.69 x 0.85 x 6.25 / 1.5+2.4 x 69.95 / 1.15

= 1409.44

EI_{eff} = 29000 x 49.1+29000 x 53.2+3750 x 3152.2

= 14,787,575

$$P_E = \frac{(EI)_e \pi^2}{(KL)^2} = (14{,}787{,}575) \times \pi^2/68^2$$

= 31563 kips

$$\lambda = \sqrt{\frac{A_s F_y + 0.85 A_c f_c' + A_r F_{yr}}{P_E}} \leq 2.0$$

$$= \sqrt{\frac{11.91 \times 57.26 + 0.85 \times 181.69 \times 6.25 + 2.4 \times 69.95}{31563}} = 0.24$$

$$f_k = \frac{1 - \alpha(\lambda - 0.2) + \lambda^2}{2\lambda^2} = \frac{1 - 0.49(0.24 - 0.2) + 0.24^2}{2(0.24)^2} = 9.31$$

$$\kappa = f_k - \sqrt{f_k^2 - \frac{1}{\lambda^2}} = 9.31 - \sqrt{9.31^2 - \frac{1}{0.24^2}} = 0.986 \leq 1.0$$

$P_u = \kappa P_{Pl}$ =0.986 x 1409.44 = 1389.5 kips

Design Example 2: Circular concrete filled composite column

Diameter of tube:	6.5 in.
Thickness of tube:	0.111 in.
Concrete strength:	15.66 ksi
Steel yield strength:	52.7 ksi
Modulus of steel:	29000 ksi
Steel area:	2.23 in.^{2}
Concrete area:	30.92 in.^{2}
I_c =	1216.95 in.^{4}
I_s =	90.79 in.^{4}
Modulus of concrete:	7270 ksi
Length of column:	22.7 in.

Design by AISC 1999:

From Chapter I

$$F_{my} = F_y + c_1 F_{yr} (A_r / A_s) + c_2 f_c^{'} (A_c / A_s)$$

$$= 52.7 + 0.85 \times 15.66 \times (30.92 / 2.23)$$

$$= 237.5$$

$$E_m = E + c_3 E_c (A_c / A_s)$$

$$= 29000 + 0.4 \times 7270 \times (30.92 / 2.23)$$

$$= 69369$$

$$r_m = \sqrt{D^2 + (D - 2t)^2} / 4 = \sqrt{6.5^2 + (6.5 - 2 \times 0.111)^2} / 4$$

$$= 2.26 \text{ in.}$$

From Chapter E:

$$\lambda_m = \frac{kl}{r_m \pi} \sqrt{\frac{F_{my}}{E_m}}$$

$$\lambda_m = \frac{22.7}{2.26\pi} \sqrt{\frac{237.5}{69369}}$$

$$=0.188$$

$$P_u = \phi_c P_n = \phi_c A_s F_{cr} = \phi_c A_s (0.658^{\lambda^2})$$

$$= 0.85 \text{ x } 2.23 \text{ x } 234$$

$$=442.9 \text{ kips}$$

Design by Proposed AISC 2005:

$$C_3 = 0.6 + 2\left(\frac{A_s}{A_c + A_s}\right) \leq 0.9$$

$$= 0.6 + 2\left(\frac{2.23}{30.92 + 2.23}\right) = 0.7 \leq 0.9$$

$EI_{eff} = E_s I_s + 0.5\ E_r I_{sr} + C_3\ E_c I_c$

$= 29000 \text{ x } 90.79 + 0.7 \text{ x } 7270 \text{ x } 1216.95$

$= 735{,}307$

$$P_0 = A_s F_y + A_{sr} F_{yr} + C_2 f_c^{'}$$

$= 2.23 \text{ x } 52.7 + 0.95 \text{ x } 15.66$

$= 577.3$ kips

$$P_E = \frac{(EI)_e \pi^2}{(KL)^2}$$

$$= \frac{(735{,}307) \times \pi^2}{(22.7)^2}$$

$=14039$ kips

$$\alpha = \sqrt{P_o / P_e} = \sqrt{577.3/14039} = 0.2$$

$$P_u = \phi_c P_n = \phi_c P_o (0.658^{\alpha^2})$$

$$= 0.75 \times 577.3 \times (0.658^{0.2^2})$$

$=425.6$ kips

Design by Eurocode 4:

E_{ce} = 600 x 15.66 = 9396 ksi

EI_{eff} = 29000 x 90.79 + 7270 x 1216.95

= 1,043,880

$$P_E = \frac{(EI)_e \pi^2}{(KL)^2} \quad = (1{,}043{,}880) \times \pi^2/22.7^2$$

= 19930 kips

γ_s = 1.1

γ_c = 1.5

γ_r = 1.15

$$\lambda = \sqrt{\frac{A_s F_y + 0.85 A_c f_c' + A_r F_{yr}}{P_E}} \leq 2.0$$

$$= \sqrt{\frac{2.23 \times 52.7 + 0.85 \times 30.92 \times 15.66}{19930}} = 0.174$$

$$\eta_{10} = 4.9 - 18.5\lambda + 17\lambda^2 = 2.2$$

$$\eta_1 = \eta_{10} + (1 - 10\frac{\text{e}}{\text{d}}) = 2.2$$

$$\eta_{20} = 0.25(3 + 2\lambda) = 0.84$$

$$\eta_2 = \eta_{20} + (1 - \eta_{20})10\frac{\text{e}}{\text{d}} = 0.84$$

$$P_{pl} = A_s \frac{F_y}{\gamma_s}\eta_2 + A_c \frac{f_c'}{\gamma_c}(1 + \eta_1 \frac{t}{d}\frac{F_y}{f_c'}) + A_r \frac{F_{yr}}{\gamma_r}$$

$$= 2.23\frac{52.7}{1.1}0.84 + 30.92\frac{15.66}{1.5}(1 + 2.2\frac{0.111}{6.5}\frac{52.7}{15.66})$$

$= 452.8$ kips

$$f_k = \frac{1-\alpha(\lambda-0.2)+\lambda^2}{2\lambda^2} = \frac{1-0.21(0.174-0.2)+0.174^2}{2(0.174)^2} = 16.98$$

$$\kappa = f_k - \sqrt{f_k^2 - \frac{1}{\lambda^2}} = 16.98 - \sqrt{16.98^2 - \frac{1}{0.174^2}} = 1.0 \le 1.0$$

$P_u = \kappa P_{Pl} = 1 \text{ x } 452.8 = 452.8$ kips

Design Example 3: Rectangular concrete filled composite column

Overall dimension:	7.87 in. x 7.87 in..
Thickness of tube:	0.354 in.
Concrete strength:	3.59 ksi
Steel yield strength:	52.8 ksi
Modulus of steel:	28043 ksi
Steel area:	10.66 in.^{2}
Concrete area:	51.34 in.^{2}
I_c =	219.67 in.^{4}
I_s =	100.67 in.^{4}
Modulus of concrete:	3481 ksi
Length of column:	23.6 in.

<u>Design by AISC 1999:</u>

<u>From Chapter I</u>

$$F_{my} = F_y + c_1 F_{yr}(A_r / A_s) + c_2 f_c^{'}(A_c / A_s)$$

$$= 52.8 + 0.85 \times 3.59 \times (51.34/10.66)$$

$$= 67.5$$

$$E_m = E + c_3 E_c (A_c / A_s)$$

$$= 28043 + 0.4 \times 3481 \times (51.34/10.66)$$

$$= 34751$$

$$r_m = \sqrt{\frac{b^4 + (b-2t)^4}{12A_s}} = \sqrt{\frac{7.87^4 + (7.87 - 2 \times 0.354)^4}{12 \times 10.66}}$$

$$= 3.07 \text{ in.}$$

<u>From Chapter E:</u>

$$\lambda_m = \frac{kl}{r_m \pi}\sqrt{\frac{F_{my}}{E_m}}$$

$$\lambda_m = \frac{23.6}{3.07\pi}\sqrt{\frac{67.5}{34751}}$$

$$=0.108$$

$$P_u = \phi_c P_n = \phi_c A_s F_{cr} = \phi_c A_s (0.658^{\lambda^2})$$

$$= 0.85 \text{ x } 10.66 \text{ x } 67.2$$

$$= 608.4 \text{ kips}$$

Design by Proposed AISC 2005:

$$C_3 = 0.6 + 2\left(\frac{A_s}{A_c + A_s}\right) \le 0.9$$

$$= 0.6 + 2\left(\frac{10.66}{51.34 + 10.66}\right) = 0.944 \le 0.9$$

$=0.9$

$EI_{eff} = E_s I_s + 0.5\ E_r I_{sr} + C_3\ E_c I_c$

$= 28043 \text{ x } 100.67 + 0.9 \text{ x } 3481 \text{ x } 219.67$

$= 3{,}511{,}215$

$$P_0 = A_s F_y + A_{sr} F_{yr} + C_2 f_c^{'}$$

$= 10.66 \text{ x } 52.8 + 0.85 \text{ x } 3.59$

$= 719.23$ kips

$$P_E = \frac{(EI)_e \pi^2}{(KL)^2}$$

$$= \frac{(3{,}511{,}215) \times \pi^2}{(23.6)^2}$$

$=62104$ kips

$$\alpha = \sqrt{P_o / P_e} = \sqrt{719.23 / 62104} = 0.108$$

$$P_u = \phi_c P_n = \phi_c P_o (0.658^{\alpha^2})$$

$$= 0.75 \times 719.23$$

$=536.82$ kips

Design by Eurocode 4:

$E_{ce} = 600 \times 3.59 = 2154.56$ ksi

$EI_{eff} = 28043 \times 100.67 + 3481 \times 219.67$

$= 3,296,246$

$$P_E = \frac{(EI)_e \pi^2}{(KL)^2} = (3,296,246) \times \pi^2/23.6^2$$

$= 58302$ kips

$\gamma_s = 1.1$

$\gamma_c = 1.5$

$\gamma_r = 1.15$

$$\lambda = \sqrt{\frac{A_s F_y + 0.85 A_c f_c' + A_r F_{yr}}{P_E}} \le 2.0$$

$$= \sqrt{\frac{10.66 \times 52.8 + 0.85 \times 51.34 \times 3.59}{58302}} = 0.113$$

$$P_{pl} = A_s \frac{F_y}{\gamma_s} + A_c \frac{0.85 f_c'}{\gamma_c} + A_r \frac{F_{yr}}{\gamma_r}$$

$$= 10.66 \frac{52.8}{1.1} + 51.34 \frac{3.59}{1.5}$$

$= 634.3$ kips

$$\mathrm{f_k} = \frac{1 - \alpha(\lambda - 0.2) + \lambda^2}{2\lambda^2} = \frac{1 - 0.21(0.113 - 0.2) + 0.113^2}{2(0.113)^2} = 38.82$$

$$\kappa = \mathrm{f_k} - \sqrt{\mathrm{f_k^2} - \frac{1}{\lambda^2}} = 38.82 - \sqrt{38.82^2 - \frac{1}{0.113^2}} = 1.0 \le 1.0$$

$P_u = \kappa P_{Pl} = 1 \times 634.3 = 634.3$ kips

APPENDIX C

DETAILED COMPOSITE COLUMN DATABASE

Table C-1 SRC Column database for 32nd column

Properties

Col. No.	Spec. No.	Source	Orig. Report	Fy Flange	Fy Web	f'c (ksi)	Cube	Fyr (ksi)	Es (ksi)	Ac (in^2)	Ec (ksi)	Ic (in^4)	Steel Section	As (in^2)
32	B2	ST	Y		40.6	1.84	*	40	29900	16.32	2491.99	16.20	BS 3 x 1.5	1.18

XX				YY																	
I (in^4)	r (in)	S (in^3)	Z (in^3)	I (in^4)	r (in)	S (in^3)	Z (in^3)	Reinforcement Longitudinal	Reinforcement Transverse	Ar (in^2)	Ixr (in^4)	Iyr (in^4)	rxr (in)	ryr (in)	h1 (in)	h2 (in)	kl (in)	kl/r	ρss	rm (in)	Pexp (k)
1.66	1.19	1.11	1.309	0.13	0.33	0.17	0.28	None		0	0	0	0	0	5.0	3.5	64.0	193.9	6.7%	1.50	60.1

AISC 1999

ϕ	AISC 1999											
0.85	Col. No.	Spec. No.	Source	Ec (ksi)	Em (ksi)	Fym (ksi)	rm (in)	λ	Fcr (ksi)	Pn	ϕPn (k)	Pexp/Pn
	32	B2	ST	2492	36793	55.9	1.50	0.529	49.69	58.6	49.8	1.21

AISC 2005

AISC 2005											
Po	L/D	EI eff	C1	I_s	P_E	α	K	Pn	P_{exp}	P_{exp} / ϕP_n	P_{exp} / P_n
73.43	12.80	59117.91	0.23	1.66	142.45	0.72	0.81	59.18	60.10	1.35	1.02

Eurocode 4

γs	γc	γr	Eurocode															
1.10	1.50	1.15	Col. No.	Spec. No.	Source	Npl.Rd (k)	Ece (ksi)	Es (ksi)	Er (ksi)	Is (in^4)	Ic (in^4)	Ir (in^4)	(EI)e (k-in^2)	Ncr (k)	λ	fk	κ	Pn (k)	Pexp/Pn
			32	B2	ST	60.57	1104	29900	29000	1.7	16.2	0.0	67524	162.7	0.672	1.786	0.799	48.4	1.241

Table C-2 SRC Beam-Column database for 80th column

Properties

															XX				YY			
Col. No.	Spec. No.	Source	Orig. Report	Type of Bending	Fyf ksi	Fyw ksi	f'c ksi	Fyr ksi	Es ksi	Ac in^2	Ec ksi	Steel Section	As in^2	Aw in^2	I in^4	r in	S in^3	Z in^3	I in^5	r in	S in^3	Z in^3
80	27	RMS		Major		39.2	6.09	60.9	29000	118	4534	HE 200 M	20.31	5.12	255.6	3.54	59.01	69.26	87.72	2.07	21.60	33.14

														e(exp)				
Long.	Trans.	Ar in^2	Ixr in^4	Iyr in^4	rxr in	ryr in	cr in	h1 in	h2 in	kl in	kl/r	e/D	ρss	Pexp k	Major in	Major in	Minor in	Qexp k	Mmax k-in
(4) 12 mm bars	5 mm ties	0.70	16.99		4.93		1.10	11.81	11.81	485.0	233.8	0.10	0.15	939.7	1.18				1108.9

AISC 1999

ϕ Factors		AISC																								
ϕb	ϕc	Col. No.	Spec. No.	Source	Type of Bending	Ec ksi	Em ksi	Fym ksi	rm in	e in	λ	Fcr ksi	Pn k	Peuler k	Z in^3	Mp k-in	A	B	C	Pp1 k	Cm	B1	Pp k	K	Pp/Pn	Pexp/Pp
0.9	0.85																									
		80	27	RMS	Major	4534	34292	62.0	3.54	1.18	1.852	15.84	321.7	366.8	69.26	3703.85	-30001	2.0E+07	-3.0E+09	224.0			224.0	0.3	0.696	4.196

AISC 2005

AISC 2005																						
Po	L/D	EIeff	C1	Interaction Diagram Bending	I_s	P_E	α	K	P_n	P_{exp}	$P_{exp} / \phi P_n$	M_{exp}	M_p	$M_{exp} / \phi M$	P_{cb}	M_{cb}	Location	IE4-1e	P_{exp} / P_n	M_{exp} / M_p	$P_{exp} / \phi P_n$	$M_{exp} / \phi M_p$
1451.4	41.06	9494598	0.30	Major	255.63	398.38	1.91	0.24	349.38	939.75	3.59	1109	3704	0.33	91.69	4487	ABOVE	4.721	3.29	0.30	4.39	0.33

Eurocode 4

Safety Factors			Eurocode Preliminary Calculations															
γs	γc	γr	bf	h	tw	tf	Ic	Is	Npl.Rd	(EI)e	Ncr	λ	fk	κ	NSd	Npm.Rd	Wpr	Wpc
1.1	1.5	1.15	in	in	in	in	in^4	in^4	k	k-in^2	k				k	k	in^3	in^3
fyd	fcd	frd																
35.6	3.45	53.0	8.661	8.110	0.591	0.984	1349.1	255.63	1169.0	12836673	538.6	1.642	0.776	0.295	344.8	613.6	3.36	339.28

Solve for Location of Neutral Axis																						
Web			Flange			Outside																
hn	Wpsn	Y/N	hn	Wpsn	Y/N	hn	Wpsn	Y/N	hn	Wpsn	Mmax.Rd	Wpcn	Mpn.Rd	Mpl.Rd	Mexp /	Pexp /	NE.Rd	hE	WpsE	WpcE	ΔME.Rd	ME.Rd
in	in^3		in	in^3		in	in^3		in	in^3	k-in	in^3	k-in	k-in	Mpl.Rd	Npl.Rd	k	in		in^3	k-in	k-in
3.80	8.52	N	3.16	10.62	Y	-9.35	69.26	N	3.16	10.62	3229.6	107.65	563.9	2665.6	0.416	0.804	N/A	N/A	N/A	0.000	N/A	N/A

Interaction Diagram Points										Interaction Diagram Lines									
A		E		C		D		B		A-E		E-C		A-C		C-D		D-B	
x	y	x	y	x	y	x	y	x	y	Slope	Intercept	Slope	Intercept	Slope	Intercept	Slope	Intercept	Slope	Intercept
0.00	1.0	0.500	0.762	1.000	0.525	1.212	0.262	1.0	0.0	-0.475	1.000	-0.475	1.000	-0.475	1.0	-1.240	1.8	1.240	-1.2

Plastic Force-Moment Interaction Point													
Load	Load Line-Interaction Diagram Intersection Points					Location of Intersection					Pcalc	Mcalc	Pexp/
Slope	Load,AE	Load,EC	Load,AC	Load,CD	Load,DB	AE	EC	AC	CD	DB	k	k-in	Pcalc
1.932	N/A	N/A	0.80	1.08	-3.46	N/A	N/A	Y	N	N	938.3	1107.2	1.002

Table C-3 CCFT column database for 88th column

Properties

Col. No.	Source	Spec. No.	Section dia.	Section thick.	D/t	Fy (ksi)	Es	Is	f'c (ksi)	Ec	Ic	Local Buckling	Cube	w (pcf)	Ac (in^2)	As (in^2)	ρss	t (in)	kl (in)	rm (in)	kl/r	Pexp (k)
88	SSKP70	30F	2.000	0.065	30.8	76.0	29000	0.19	4.04	3693	9.60	OK		150	2.75	0.395	0.13	0.065	42.0	0.685	61.4	27.1

AISC 1999

ϕ	AISC									
0.85	Col. No.	Source	Ec (ksi)	Em (ksi)	Fym (ksi)	rm (in)	λ	Fcr (ksi)	Pn (k)	Pexp/Pn
	88	SSKP70	3693	39266	99.9	0.685	0.985	66.5	22.3	1.21

AISC 2005

AISC 2005											
Po	L/D	EI eff	C1	I_s	P_E	α	K	Pn	P_{exp}	$P_{exp} / \phi P_n$	P_{exp} / P_n
40.6	21.0	7256.6	0.9	0.2	40.6	1.0	0.7	26.7	27.1	1.4	1.0

Eurocode 4

Safety Factors			European (Roik and Bergmann)																				w/out Confinement	
γs	γc	γr	Local Buckling ε	d/t	Check	Ece (ksi)	Es (ksi)	Ic (in^4)	Is (in^4)	(EI)e (k-in^2)	Ncr (k)	λ	η10	η20	η1	η2	Npl.Rd (k)	Confinement Factor	fk	κ	NSd. (k)	Pexp/NSd.	NSd. (k)	Pexp/NSd.
1.10	1.50	1.15																						
69.1	2.69		0.67	30.8	OK	2424	29000	0.600	0.185	6824	38.2	1.038	0.00	1.00	0.00	1.00	34.7	0.84	1.05	0.64	22.2	1.22	22.2	1.22

Table C-4 CCFT beam-column database for 2th column

Properties

Col. No.	Spec. No.	Source	Fy (ksi)	Es (ksi)	f'c (ksi)	Ec	Ac (in^2)	As (in^2)	Z (in)	d (in)	ts (in)	D/t	e/D	ρss	Local Buckling	kl (in)	rm (in)	kl/r	Pexp (k)	e(exp) (in)	Mexp k-in
2		F67	60.0	29000	4.20	3765.0	14.17	1.730	2.393	4.50	0.125	36.0	0.26	10.9%	OK	5.0	1.547	3.231	90.0	1.178	106.0

AISC 1999

ϕb	ϕc	AISC																		
0.90	0.85	Col. No.	Source	Ec (ksi)	Em (ksi)	Fym (ksi)	rm (in)	λ	Fcr (ksi)	Pn (k)	Peuler (k)	Mp (k-in)	A	B	C	D	Pp1 (k)	Pp2 (k)	Pp (k)	Pp/Pn vs. 0.2	Pexp/Pp
		2	F67	3765	41335.2	89.2	1.547	0.048	89.2	154.2	67599.9	143.59	-1163	1.6E+08	-1E+10		63.5	139462.3	63.5	0.412	1.42

AISC 2005

AISC 2005																						
			Interaction Diagram																			
Po	L/D	EI eff	C1	I_s	P_E	α	K	Pn	P_{exp}	$P_{exp}/\phi P_n$	M_{exp}	M_p	$M_{exp}/\phi M_p$	P_{cb}	M_{cb}	Location	IE4-1e	P_{exp}/P_n	M_{exp}/M_p	$P_{exp}/\phi P_n$	$M_{exp}/\phi M_p$
160.34	1.11	168602.9	0.82	4.11	66561.74	0.05	1.00	160.18	90.00	0.75	106.00	143.59	0.82	23.71	148.62	ABOVE	1.468	0.486	0.738	0.65	0.82

Eurocode 4

Safety Factors																			
γs	γc	European (Roik and Bergmann)																	
1.10	1.50	Local Buckling			Ece	Es	Ic	Is	(EI)e	Ncr	λ	η10	η20	η1	η2	Npl.Rd.	fk	κ
fyd	fcd	ε	d/t	Check	(ksi)	(ksi)	(in^4)	(in^4)	(k-in^2)	(k)						(k)		
54.5455	2.80	0.75	36.0	OK	2520.0	29000	16.0	4.1	159662.1	63032	0.1	4.0	0.8	0.0	1.0	134.0	187.4	1.0

NSd. (k)	Npm.Rd	Wpc	hn	Wpcn	Wpsn	hE	NERd	MmaxRd	MpnRd	MplRd	WpcE	WpsE	ΔMERd	ME.Rd
134	39.7	12.8	0.5	1.1	0.1	1.4	73.8	148.5	5.0	143.4	8.1	0.5	37.2	111.2

Eurocode 4

Interaction Diagram Points										Interaction Diagram Lines							
A		E		C		D		B		A-E		E-C		C-D		D-B	
x	y	x	y	x	y	x	y	x	y	Slope	Intercept	Slope	Intercept	Slope	Intercept	Slope	Intercept
0	1	0.776	0.551	1	0.296	1.035	0.148	1	0	-0.579	1.00	-1.136	1.43	-4.226	4.52	4.226	-4.23

Predictions Using First Order Moments														
Mexp / Mpl	Pexp / Npl	Load Slope	d Line-Interaction Diagram Intersection Po				Location of Intersection				Pcalc	Mcalc	Pexp/
			Load,AE	Load,EC	Load,CD	Load,DB	AE	EC	CD	DB	k	k-in	Pcalc
0.739	0.671	0.90853	0.611	0.636	0.800	1.157	Y	N	N	N	81.9	96.4	1.099

Table C-5 RCFT column database for 43th column

Properties

Col. No.	Source	Spec. No.	Section b(in)	Section t(in)	Fy (ksi)	Es (ksi)	Is	Ec	Ic	f'c (ksi)	w (pcf)	Ac (in^2)	As (in^2)	Local Buckling	ρss	h1 (in)	h2 (in)	t (in)	b/t	kl (in)	rm (in)	kl/r	Pexp (k)
43	BNMF95	CR6-A-4-2	5.673	0.250	89.4	29000	26.68	4448	59.65	5.86	150	26.75	5.431	OK	0.17	5.67	5.67	0.250	22.7	8.51	2.216	3.84	621.5

AISC 1999

AISC										
φc 0.85	Col. No.	Source	Ec (ksi)	Em (ksi)	Fym (ksi)	rm (in)	λ	Fcr (ksi)	Pn (k)	Pexp/Pn
	43	BNMF95	4448	37764	114.0	2.216	0.067	113.8	525.2	1.184

AISC 2005

AISC 2005											
Po	L/D	EI eff	C3	I_s	P_E	α	K	Pn	P_{exp}	P_{exp} / ϕP_n	P_{exp} / P_n
618.996	1.500	1012416.93	0.900	26.677	137979.99	0.067	0.998	617.835	621.528	1.341	1.006

Eurocode 4

		European (Roik and Bergmann)														
γs 1.10	γc 1.50	Local Buckling ε	b/t	Check <=52ε	Npl.Rd (k)	Ece (ksi)	Es (ksi)	Ic (in^4)	Is (in^4)	(EI)e (k-in^2)	Ncr (k)	λ	fk	κ	NSd. (k)	Pexp/NSd.
		0.617	22.66	OK	546.1	3517.27	29000	59.65	26.68	983432	134029.7	0.06924	101.94	1.0000	546.1	1.138

Table C-6 RCFT beam-column database for 29th column

Properties

Col. No.	Source	Spec. No.	Type of Bending	Fy ksi	Es	Is	f'c ksi	Ec	Ic	Ac in^2	As in^2	Z in^3	h1 in	h2 in	ts in	kl in	kl/r	rm in	b/t	ρss	e/b	Pexp k	e(exp) x in	e(exp) y in	Mmax k-in
29	G93	1		44.1	29000.0	6.5	6.82	4796.44	35.02	18.76	3.57	6.06	4.72	4.72	0.197	245.0	132.4	1.85	24.0	16.0%	0.17	137.1	0.79		108.0

AISC 1999

ϕb	ϕc																				
0.90	0.85	AISC																			
		Col. No.	Source	Local Buckling	Ec (ksi)	Em (ksi)	Fym (ksi)	rm (in)	λ	Fcr (ksi)	Pn (k)	Peuler (k)	Mp (k-in)	A	B	C	D	Pp1 (k)	Pp (k)	Pp/Pn	Pexp/Pp
		29	G93	OK	4796	39093	74.6	1.85	1.841	19.3	68.8	78.4	267.0	-2163.0	325026	-1E+07		42.6	42.6	0.619	3.219

AISC 2005

AISC 2005

Po	L/D	EI eff	Interaction Diagram C1	I_s	P_E	α	K	Pn	P_{exp}	$P_{exp}/\phi P_n$	M_{exp}	M_p	$M_{exp}/\phi M_p$	P_{cb}	M_{cb}	Location	IE4-1e	P_{exp}/P_n	M_{exp}/M_p	$P_{exp}/\phi P_n$	$M_{exp}/\phi M_p$
265.85	51.86	480416.2	0.9	12.2	79.0	1.8	0.3	69.3	137.1	2.64	108.0	267.0	0.45	25.88	267.0	ABOVE	3.868	2.56	0.40	3.42	0.45

Eurocode 4

Safety Factors															
γs	γc	European (Roik and Bergmann)													
1.1	1.50	Local Buckling ε	d/t	Check	Ece (ksi)	Es (ksi)	Ic (in^4)	Is (in^4)	(EI)e (k-in^2)	Ncr (k)	λ	Npl.Rd. (k)	fk	κ	
fyd	fcd														
40.1	4.54	0.879	24.0	OK	4089.92	29000.0	29.3	12.2	473766	77.9	1.913	228.1	0.69	0.242	

NSd. (k)	Npm.Rd	Wpc	hn	Wpcn	Wpsn	hE	NERd	MmaxRd	MpnRd	MplRd	WpcE	WpsE	ΔMERd	ME.Rd
55.2	85.2	20.3	0.83	3.00	0.27	1.60	124.4	288.9	17.7	271.2	11.0	1.0	65.3	223.6

Eurocode 4

Interaction Diagram Points										Interaction Diagram Lines							
A		E		C		D		B		A-E		E-C		C-D		D-B	
x	y	x	y	x	y	x	y	x	y	Slope	Intercept	Slope	Intercept	Slope	Intercept	Slope	Intercept
0	1	0.824	0.546	1	0.374	1.065	0.187	1	0	-0.551	1.00	-0.979	1.35	-2.859	3.23	2.859	-2.86

Predictions Using First Order Moments													
Mexp / Mpl	Pexp / Npl	Load Slope	d Line-Interaction Diagram Intersection Po				Location of Intersection				Pcalc k	Mcalc k-in	Pexp/ Pcalc
			Load,AE	Load,EC	Load,CD	Load,DB	AE	EC	CD	DB			
0.40	0.60	1.51	0.73	0.82	1.12	3.20	Y	N	N	N	167.1		0.821

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